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Construction of Lethbridge Viaduct Canadian Pacific Railway

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The Crow's Nest line of the Canadian Pacific Railway Company leaves the main transcontinental line at Coleridge, about seven miles east of Medicine Hat, Alberta, and it extends westerly through the mountains, traversing the rich mining regions of Southern British Columbia, about 100 miles south of the main line.

The old line between Lethbridge Junction and Macleod was 37 miles long, and Lethbridge was reached by a spur track of one and a half miles, making the total distance from Lethbridge to Macleod 38½ miles. This line had curves as sharp as 7 degrees (819 ft. radius) and an actual grade of 1 per cent (52.8 ft. per mile). No compensation was allowed for curvature, so that the ruling gradient was equal to a virtual 1.2 per cent grade (63.4 ft. per mile).

On this old line, which was constructed in 1897-98, there were 20 bridges, having an aggregate length of 12,063 ft. or 2.8 miles; 1,450 ft. of this length consisted of Howe truss spans, varying in length from 100 ft. to 150 ft. One of the bridges was 2,933 ft. long, and a number of them were over 100 feet high. In many cases they crossed streams and ravines with very steeply cut banks, which required constant watching, especially during the spring and high water seasons, owing to the nature of the soil. The life of these bridges having very nearly expired, they would have to be rebuilt at an estimated cost of $1,065,000 if the old line were to be maintained.

In consideration of these circumstances, the rapid increase in traffic, and the fact that the original charter of the Crow's Nest branch required the construction of a line from Lethbridge, it was decided to have surveys made to ascertain if it would be possible to secure a line between Lethbridge and Macleod having lower grades and better alignment.

With this object in view, extensive surveys were made in 1904 and 1905, with the result that a line was finally adopted. This line gives a virtual maximum grade of four-tenths of 1 per cent (21.12 ft. per mile), with a maximum curvature of three degrees (1,910 ft. radius).

The saving effected by the new location, as compared with the old, is as follows: 5.26 miles of line; 1,735 degrees of curvature (eliminating 37 curves), and 401.5 ft. less rise and fall, thereby securing a 0.4 per cent grade, as compared with the 1.2 per cent grade on the old line.

On the supposition of an increase of 20 per cent in traffic above what it was the last year, these changes should so reduce operating expenses as to pay interest at the rate of 4 per cent on an investment of $3,625,000. If to this amount there is added the cost of replacing old bridges with permanent structures, viz., $1,065,000; a total of $4,690,000 would be obtained. This amount represents the capital expenditure which would be justified, while the estimated cost of the change of line was $2,048,700.

Fig. 1.—General View, Showing Structure in Progress.

On the new line there are only two bridges, both large steel viaducts, one over the Belly River at Lethbridge, 5,327 ft. long, with a maximum height of 314 ft. from the bed of river to the base of rail; and the other over the Old Man River Valley, 1,900 ft. long and 146 ft. high. The former structure is the subject of this paper. The east end of the viaduct is 3,800 ft. west of Lethbridge Station.

Lethbridge is in the center of an irrigation tract, and also of an important coal mining district. It is a divisional point of the Crow's Nest branch (Canadian Pacific Railway), and the terminus of the Alberta Railway & Irrigation Company's railway. It is situated in the Province of Alberta, about 750 miles west

*From a paper presented before the Canadian Society of Civil Engineers.

of Winnipeg and 2,174 miles west of Montreal. Its population, which is increasing rapidly, is at present about 6,000.

The site of the bridge is the best for a high-level crossing in the immediate vicinity of the town. On the east side the surface slopes are fairly uniform for a distance of 1,000 ft. from the prairie level, when they drop more abruptly for 800 ft. to the flat at the bottom of the valley, which is flooded in extreme high water. The valley is approximately level for a distance of 1,800 ft., to the edge of the river, which at the crossing is about 300 ft. wide. On the west side of the river the ground rises to the prairie level in a distance of about 1,300 ft. On the east bank the soil is clay and gravel for about 6 ft. below the surface, then 50 ft. of hard clay, followed by 12 to 20 ft. of coarse gravel, and, below this, shale and coal. For about 20 ft. below the bed of the river there is gravel, then 20 ft. of coal shale, below that 2 ft. of blue clay, then a hard shale for a distance of 16 to 18 ft. below which is hardpan and sandstone. The west side is of somewhat similar formation, but the bank has been eroded, causing it to cave in at several points. On this account it was necessary to do extensive work in order to secure proper foundations for the piers.

On the east side there are some old mine workings which follow a vein of coal averaging 7 ft. thick. The vein is practically horizontal, at the same elevation as the flat alongside the river, and varies in depth below the surface of the ground from 25 ft. at bent No. 23 to 150 ft. near the east end of the bridge.

When the site had been determined borings were made at various points along the bridge line, giving the results above mentioned. Several test pits were sunk and test piles of fir about 11 in. in diameter were driven to a depth of 24 ft. These were each loaded with 30 tons, and after one month showed practically no settlement.

The alignment of the bridge is a tangent throughout. At the eastern approach there is a 1 degree curve and a 3 degree curve at the western approach. The grade is 0.4 per cent, rising towards the west for the whole length of the bridge.

Design.—Numerous preliminary studies were made in order to determine the most economical type of bridge, and the design finally decided upon was a steel viaduct consisting of 44 plate girder spans 67 ft. 1 in. long, 22 plate girder spans 98 ft. 10 in. long, and one riveted deck lattice truss span 167 ft. long, carried on 33 rigidly braced riveted steel towers.

For the substructure it was decided to use concrete piles supporting concrete pedestals. It was not considered advisable to use wooden piles, as the foundations would be alternately wet and dry. All the land piers, with the exception of six, were built on concrete piles, the excavations averaging 7 ft. deep, but in some cases running as deep as 18 ft. The length of the concrete piles varied from 12 to 20 ft. The type used was the Raymond tapered pattern, which consists of a sheet-iron casting, inside of which is a collapsible core, the latter being withdrawn when the pile is driven and the casing then filled with concrete. Tapered holes, 2 ft. 2 in. square at the bottom and 18 in. at the top, as shown on plate 8, were formed in the piers to receive the anchor bolts. After the piers were completed, anchor bolts were set in place accurately as to centers and height, and grouted in for about half the length of the bolts, the balance of the bolt being left free so that it could be sprung, if necessary, when erecting the steelwork. The tops of these holes projected far enough beyond the edge of the bed plates to enable them to be completely filled with grout on completion of the structure.

The high bents, Nos. 23 to 56, inclusive, were provided with four anchor bolts 2½ in. in diameter in the foot of each column, and the remainder of the bents with two anchor bolts at each column.

The towers were designed with a batter of one to six, which, with the girders of the spans spaced at 16 ft. centers, gives ample spread at the base of the towers to keep the maximum

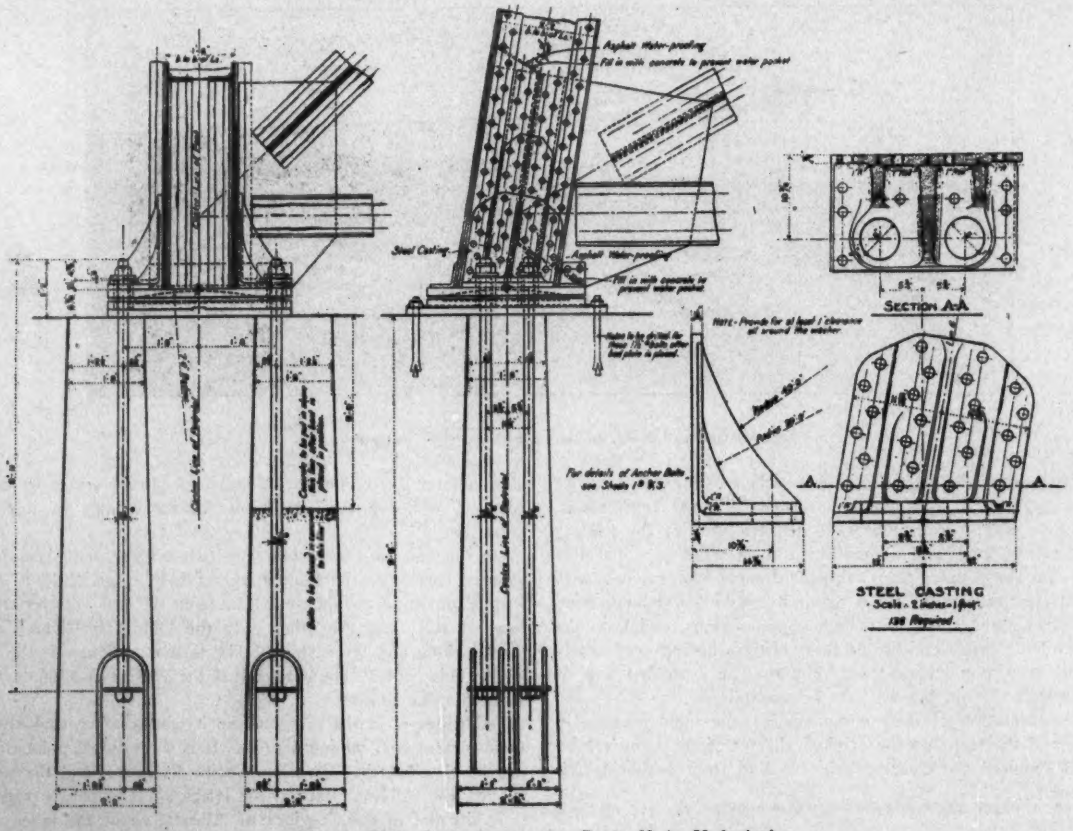


Fig. 2.—Plate 8—Anchorage for Bents 23 to 56 Inclusive.

uplift due to horizontal forces, when unloaded, within reasonable limits. The tower spans were made 67 ft. long in order to give longitudinal stiffness to the towers and reduce the traction stress in the lower legs.

The stresses in the towers were calculated to provide for four conditions, viz.: Case 1—With the bridge unloaded, wind at an angle of 45 degrees to center line of track at 50 lbs. to the square foot. Case 2—Bridge loaded, wind as per case 1, but at 30 lbs. per square foot. Case 3—Bridge unloaded, wind normal to center line of track at 50 lbs. per square foot. Case 4—Bridge loaded as per case 3, but at 30 lbs. per square foot.

The bridge was designed to meet the requirements of the Canadian Pacific Railway Company's specifications for steel bridge for 1903. The material used was mild, open hearth steel of 60,000 lbs. ultimate strength. The live load provided for consisted of two consolidation engines coupled together, weighing each 337,000 lbs., followed by a uniform train load of 4,000 lbs. per lin. ft. The effect of impact and vibration was provided for by the well-known Schneider formula:

$$I = S\left(\frac{300}{L+300}\right)$$

Where I = impact to be added to live load strain.
S = calculated maximum live load strain.
L = length of loaded distance in feet which produces the maximum strain in member.

The unit strains used in the design were as follows: Axial tension 16,000 lbs., and axial compression in the gross section $16{,}000 - 70\frac{l}{r}$, where l = length in feet and r = least radius of gyration in inches.

Allowance in the column stresses was made for 70 per cent of traction and longitudinal component of wind on train, minus stress due to a frictional resisting force due to the 100-ft. spans of adjacent towers of 10,000 lbs., corresponding to a train covering about five towers. The units were increased 25 per cent when wind was included, and 50 per cent when both wind and traction were included.

It will be noted that liberal sized sections were used in the tower legs. These were of the trough section form, with cover plates varying from 26 in. x 7/16 in. for the two upper stories to 26 in. x 11/16 in. for the bottom story, in addition to which plates and angles were added to the columns in the three bottom stories, on the opposite side to the cover plate.

The diagonal bracing, both transverse and longitudinal, are stiff riveted members composed of angles and channels latticed, and long members are supported at the center. Diagonals were figured for tension only. The use of the double diagonal type of bracing without horizontal struts was considered, but not adopted, as it was not economical, and also on account of the great length of compression members that would be required. Owing to the uplift from the wind it was necessary to design an adequate means of anchoring the column footings to the piers. The detail adopted consisted of steel castings on each side of the column rivetted thereto, and to the shoe plate, engaging 2½-in. anchor bolts, as shown on plate 8. This construction was used only for the bases of tall bents, Nos. 23 to 56, inclusive. C. P. R. standard spherical bearings were used under each column. These consist of a cast-steel disc plate planed to a flat bearing on the bottom, the top fitting into a concave socket in the shoe plate. The bottom of the disc plate is free to move on the bed plate, to allow for expansion and contraction, and also to provide for any variation from true level in the top of the concrete pedestal. The tops of the concrete pedestals were carefully dressed off before the steel work was erected. Graphite was placed between the surfaces of the bed plate and discs to assist the latter in moving. To provide for the expansion and contraction in the tower footings, one corner only of the tower was fixed, and the other three left free to move. Two of them had ribs planed in the direction of motion, and the third a plain surface. The anchor-belt holes in the shoe and disc plates were 5 in. in diameter, and at the fixed point the holes around the anchor bolts were filled with cast-iron spools. At each of the other points ¼-in. thick wrought iron spools were placed over the anchor bolt between the bed plate and the washer under the nut, so that when the nuts were screwed down these spools provided sufficient clearance between the shoe plate and the washer to prevent binding. At the tops of the towers the 67-ft. spans were fixed at both ends, as contraction on these spans would be the same as contraction of the longitudinal tower bracing. Provision for expansion at one end of each of the intermediate spans was made in the bearings on the column caps. These bearings consisted of a shoe plate with a flat key rivetted on, transverse to the center line of the girder and fitting into a socket in the cap plate of the column. The keys have 1½ in. flat bearing on the bottom, and were bevelled towards each side so as to prevent any possibility of binding on the cap plate. The slots in the latter at the expansion point were made wider than the key, so as to provide for motion, while those at the fixed points were made so as to fit the key snugly. Four anchor bolts were placed at each bearing to secure the shoes to the cap plates.

One hundred foot intermediate spans were decided upon for that portion of the structure over 125 ft. high, this being the longest span which it was thought feasible to handle from an erection standpoint, although theoretical economy would call for longer spans. On account of the severe winds experienced in this region and considering the great height of the structure it was decided to use spans of the through instead of deck-plate girder type, so as to render it practically impossible for derailed cars to leave the deck. A car in falling from the structure might knock out the tower legs and bracing, owing to the great spread of the bents, and thus cause great expense and delay to traffic.

This type of span increased the cost of the structure, but for the reasons above mentioned, in order to provide greater security to traffic, it was considered that the additional expenditure was justified. The girders of the 67 ft. 3 in. and the 100-ft. spans are of the same depth throughout, namely, 8 ft. ½ in. back to back of flange angles, and are spaced 16 ft. center to center. The track is carried on 8 in. x 12 in. ties, spaced 12 in. centers, which are supported by two lines of steel stringers, spaced 8 ft. center to center. The floor beams in all the spans are spaced 16 ft. 5¼ in. apart throughout the structure, so as to simplify the details, and all stringers were therefore made alike.

All of the spans from the east abutment to bent No. 16, and from bent No. 62 to the west abutment, were made 67 ft. 3 in. long, and 100-ft. spans were used between the towers for the remainder of the structure, with the exception of the space between bents Nos. 60 and 62. At this point there was a steeply cut bank, and it was considered inadvisable to place a tower or bent on it. A rivetted deck-lattice Warren type truss was used to span over this bank. The top chords of this span are worthy of note, in that it was necessary to build them to carry the heavy concentrated loads induced by the erection traveler. They are of box section, about 4 ft. deep, and in line with the tops of adjacent girders, the web plates extending down to the base of the rail, so that the appearance from the deck is that of a plate girder.

The C.P.R. standard inside guard rail was used on this bridge and consists of three lines of 56-lb. rail laid between the running rails. In addition to this inner guard rail, two outer guards or tie spacers of 6 in. x 8 in. timber were notched over the ends of the ties and securely bolted thereto. Hook bolts were placed in every fourth tie to secure them to the stringers.

Laying out the Work.—During the first week of December, 1907, when the design had been completed, the work of finally laying out the center and locating the position of the pedestals for the viaduct was commenced.

Permanent hubs were first erected along the center line. These

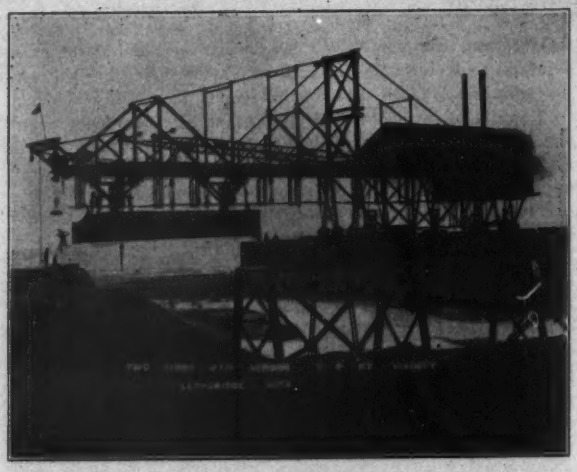


Fig. 3.—Erection of Last Span.

were located at convenient points, and were afterwards used for triangulation purposes. No attempt was made at first to locate the bents definitely, as it was considered advisable to obtain a through chainage from which the pedestals could be subsequently laid out. Owing to the fact that it was important to have the piers in the river started promptly, in order to avoid delay, these were first laid out. A base line 757 ft. long, at approximately right angles to the center line of the bridge, was laid out in the valley in which these river piers were located. The hub at the intersection of the center line and this base line was taken as the starting-point of the measurements. The base was very carefully measured several times, and the average of the different measurements taken, as it was to be used not only to locate river piers, but also as a triangulation base to check all land measurements on the steep slopes. All distances were carefully checked by triangulation in order to detect any errors in the chainage. From this base line also several other base lines were laid out and connected with the hubs located at the various points on the site, in order to check the triangulations and measurements made directly from the first base. The distances for the location of the pedestals were checked from the different hubs on the center line, both by triangulation and actual measurements.

An ordinary transit, having a 6-in. plate graduated to minutes only, was used for the triangulations. The angles were measured with the telescope in the two positions, each angle of the triangle being measured repeatedly, and the closing error divided amongst the different angles. These results gave a very close check on the chainage.

A 100-ft. steel tape, which had been compared with a tape used by the Bridge Company in the manufacture of the steel work, was at first used for the measurements. Owing to the extreme high winds which prevailed this steel tape was found inconvenient, besides which there were several inaccuracies in the minor graduations, and it was therefore decided to make the measurements with a wooden rod instead of the tape. This measuring rod consisted of a piece of pine scantling 2 in. x 4 in. x 15 ft. 10 in. long. It was carefully graduated to feet and tenths, the graduations being made on pieces of zinc 3 in. long by 1 in. wide screwed to the bottom of the rod and bevelled on the edge. The zero of the rod was a nick cut in a brass plate a few inches from the end. Throughout the operation of laying out the work this rod had to be constantly checked on a standard base, as it was found to change in length with varying conditions of atmospheric humidity. In order to establish a standard for the comparison of this rod before setting out the center line, a base was laid out with a steel tape supported over its whole length under a tension of 10 lbs. In order to do this it was found necessary to drive stakes at approximately 15 ft. apart, on which the rod could be checked under conditions similar to those used in the field. For a base line the following construction was employed: Two 8 in. x 8 in. posts were firmly placed in the ground at every 100 feet, and intermediate posts of 2 in. x 4 in. scantling about 15 feet apart were lined in with these posts and driven so that the tops were on the same level. Inch boards were nailed on edge, level with the tops of the stakes on which the tape was supported and stretched, allowance being made for temperature and error in graduation. The standard temperature adopted was 70 degrees Fahrenheit. The rod was then checked on this base by repeated trials from post to post, placing the rod on intermediate posts in a similar manner to its use in the field. The method of handling the rod and laying out the work was as follows: In the valley, from the river to the foot of the east slope, a distance of 1,600 ft., the ground was practically level. The measurements over this portion were therefore comparatively easy, stakes 2 in. x 2 in. being driven along the center approximately 15 feet apart, and their tops carefully levelled. The stakes projected above the ground surface from 3 in. to 18 in. Where the ground changed in elevation to any considerable extent the level of the tops of the stakes were raised by a step up or down to conform with the change. These steps were in some cases measured on the slope and reduced to the horizontal, or, by means of a plumb-bob, the rod being held horizontally as indicated by the level fixed to it. In order to keep the rod steady, as well as horizontal, a pointed iron rod with a siding collar and clamp attached was used. An attempt was made to employ the same method on the slope, but owing to the difficulty in holding the plumb-bob steady it was necessary to adopt other methods. The measurements were taken by placing the zero of the rod on the zero stake on a mark scratched on the head of a tack. The forward end of the rod was laid on the stake 15 ft. ahead, which was previously driven as explained above. The tack was then driven into the top of this stake to correspond with any tenth on the rod which covered the head of the stake, and the diagonal scratch made across the top of the tack with a knife blade to coincide as nearly as possible with the edge of the graduated zinc plate. The actual distance was then accurately marked by holding the graduation mark on the rod over the diagonal scratch until it exactly coincided, a cross scratch being then made at this spot and the zero end of the rod meanwhile carefully held on the zero mark. In measuring up the hill no attempt was made to obtain horizontal lengths directly. The inclined distances between hubs were observed and levels taken on each stake, from which the horizontal distance was computed.

Construction of Substructure.—The contract for the excavation and substructure was awarded to J. Gunn & Sons, of Winnipeg, work to be completed by March 1, 1908. The excavation work was started in October, 1907. Concreting was commenced in November, 1907, and carried on continuously through the winter, but owing to delays caused by extreme floods in the early spring and other difficulties met with in connection with the river foundations the substructure was not completed until February, 1909.

In June, 1908, before the excavation for the river piers was completed, there was a flood, during which the water rose one foot above the highest previous record, that of June, 1902. This high water completely submerged the coffer-dams, deposited enough silt to fill them, and carried away some of the contractors' plant.

The general formation of the river bed is gravel overlying hard shale. The average depth of water is about 10 feet. The river piers were carried down to good foundation on the hard shale. The excavation for these piers, before caissons were of 10 in. x 10 in. timber securely braced, floated into their places and sunk in proper position. Sheet piling, made of three pieces 2 in. x 10 in. plans nailed together to form a tongue and groove, was driven outside the caissons, and a second row of sheet piling driven about 3 ft. outside the first. The intervening space was filled with clay puddle. The caissons were pumped out with

some difficulty, three centrifugal pumps being required to keep the foundations clear of water. The excavation was completed with pick and shovel, and the material handled by bucket and derrick. When the excavation had reached hard shale, drillings were made to a depth of at least 20 ft. to make sure that there was no change of formation. In three of the piers it was found that the pumps could not hold the water down, and it was therefore necessary to have the bottoms carefully cleaned and prepared by divers before the concrete was deposited. A layer of concrete 4 ft. in depth was then placed in the bottom, under water, over the full area of the caisson and allowed to set, after which the caisson was pumped out and the balance of the pier finished in the dry.

Much difficulty was encountered in driving the concrete piles on the steep side-hills, and it was found necessary to make considerable alterations in the pile-driver and cribbing on which to block them. On account of the nature of the west slope it was necessary to carry down all foundations to such a depth that they would be below the natural slope of the ground if the steep banks continued to erode. The ground in the immediate vicinity of the piers was also dressed off to a 1½ to 1 slope. The toe of the west bank along the river edge was protected from erosion by constructing a reinforced concrete retaining wall for a considerable distance up and down the stream from the site of the bridge, the upstream end terminating at an exposed seam of coal shale. The foundations of this wall were carried down to hard shale and back filled, and the slopes dressed off as before mentioned. Along the flat on the east side of the river, clusters of piles were driven upstream from the towers, to catch and deflect any drift wood that might come down in flood. In addition to these piles heavy deflecting booms are to be located in the river opposite the towers for the same purpose.

Concrete and Piling.—Concrete was made in accordance with the C. P. R. standard specifications. Gravel taken from the bed of the river was used throughout.

The specification for concrete piling provided that piles should be of approved form, and vary in size according to length, all piles to be placed by first driving a core, no driving of the concrete piles themselves being allowed. The concrete forming the piles consisted of one part cement, three of sand, and six of broken stone or gravel, not exceeding ¾ in. in size. The diameters of the piling for a 30-ft. length was 20 in. at butt, and for 35 ft. to 40 ft. long, 18 in. at butt, with minimum diameter of 6 in. at point.

Erection of Steelwork.—Tenders were called for the manufacture and erection of the steelwork, and the contract awarded in October, 1906, to the Canadian Bridge Co., of Walkerville, Ont. The erection of this structure entailed much study and the installation of a large plant. A material yard was located at the west end of the railway yard and a short distance from the east end of the bridge. For the purpose of transporting material from the storage yard to the site, the railway company furnished the contractors with the constant use of a locomotive and ten flat cars. There was erected over the material yard an unloading traveller on wooden runways, as indicated on plate 13. The traveller consisted of one of the bridge company's 125 ft. highway spans, provided with end wheels, two of which at each end were connected up by gearing and shafting to a standard two-drum Beatty hoisting engine. Movement along the runway, which was 240 ft. long and 37½ ft. high, built transversely to the center line of the railway track, was

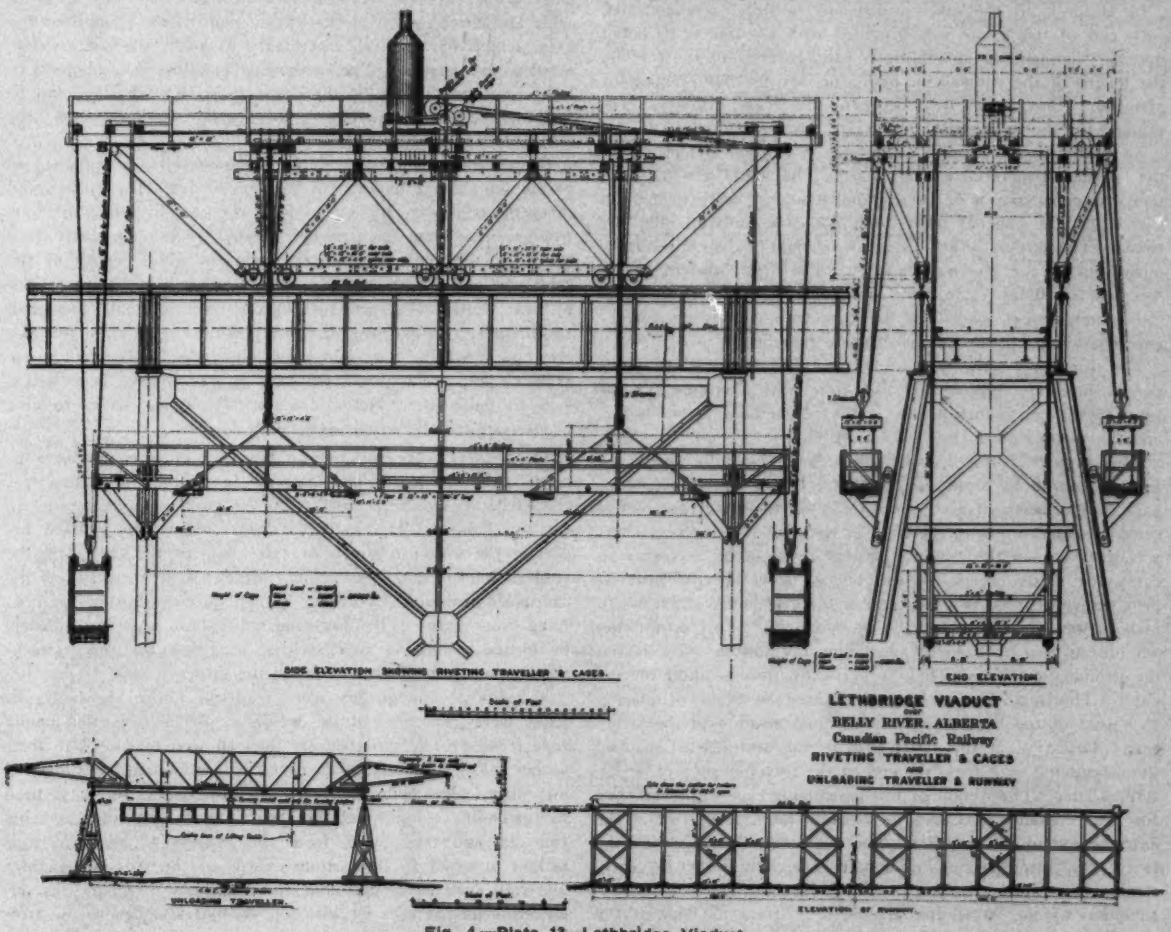


Fig. 4.—Plate 13—Lethbridge Viaduct.

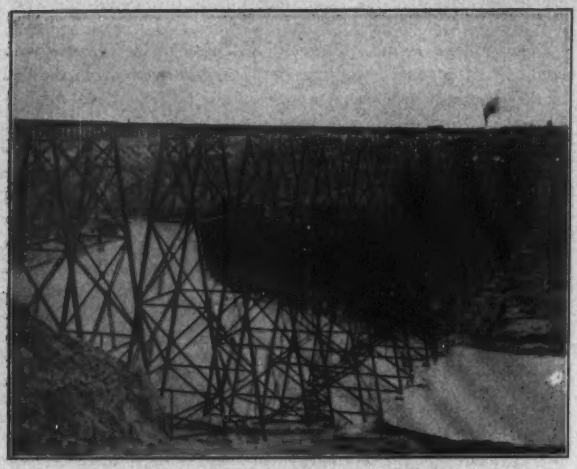


Fig. 5.—General View of Structure.

thus provided, enabling the placing of cars immediately under the traveler. Two sets of three-sheave steel blocks, rigged with ⅝-in. cable, with a capacity of 17 tons each, connected up to a standard double-drum hoisting engine located on the traveller, were provided for the purpose of lifting the 100-ft. girders, weighing about 30 tons each, off the cars. In the event of any of these girders reaching the work "wrong end to," provision was made by means of hooks and a beam arranged to swivel on the center floor beam of the 125 ft. crane to turn them end for end.

In addition to the above-mentioned hoisting arrangements, each end of this crane was provided with a 5-ton 42-ft. boom derrick, commanding an additional storage area 40 ft. wide by the length of the runways on each side, for the lighter class of material. The main area between the crane runways was utilized for the storage of girders and heavy columns.

To facilitate the work of erection, a regular program for the loading of material was arranged. The 100-ft. girders, with their accompanying floor beams and stringers, were loaded on two 80,000 lbs. capacity 40-ft. long flat cars, with an idler between. The girders were loaded at a height to be conveniently picked up by the erection traveller. The 67-ft. girders, which weighed 30,000 lbs. each, were loaded in a similar manner, and the columns were loaded on flat cars with corbals at convenient heights for lifting by the erection traveller.

The traveller for the erection of this viaduct was a very ponderous machine, built entirely of steel, with the exception of some minor parts, such as the engine-room floor, engine-house, and the various platforms.

The strains of this traveller were very carefully calculated for the conditions shown on plate 11, especial attention being paid to the wind stresses, on account of the very high winds certain to occur during the time of erection. The stress sheet was made up on the expectation that it might be necessary to carry the entire weight of the bottom story of the heaviest bent suspended 8 ft. from the end of the cantilever, which would make a total load of 90,000 lbs. at that point. This assumption was due to the intention of assembling the bottom story flat on the ground, and then lifting it vertically into position on the piers. This method was later abandoned in favor of placing each part of the bent directly in its final position in the structure. To carry out this latter plan it was desirable to suspend the assembling cage from the end of the traveller, as will be described later. The weight of this assembling cage, fully equipped for use, is twenty tons, and, as already stated, is carried at the extreme end of the traveller. The cage, with the weight of the two heavy bottom sections of columns weighing 30,000 lbs. each, strains the traveller trusses considerably above the original assumption of the 90,000 lbs. applied 8 ft. from the end of the traveller, but the original design had been made with such liberal sections that no increases were necessary, the maximum strains in tension not exceeding 14,600 lbs. per square inch, with compressive strains in proportion.

The traveller was composed of two principal trusses 207 ft. long, spaced 16 ft. apart, center to center, so as to conform to the spacing of the main girders of the spans, on the top flanges of which it was arranged to travel. The cantilever portion of the trusses had a reach of 116 ft., and the balancing portion of the trusses was 91 ft. in length. The rear end of the balancing arm was carried on a truck with three 24-in. cast-steel wheels, and the forward end on a truck consisting of four similar wheels. The rear end of the balancing arm was counter-weighted with 30 tons of steel rails in addition to the weight of the engine-house, platform, and hoisting machinery contained thereon. Special runways, consisting of oak planks framed to fit over the top cover plates of the spans, and having 80-lb. rail laid on top, were bolted together in sections, facilitating their placing and removal after each movement of the traveller. The weight of the traveller in working condition was 712,000 lbs.

In order to provide for the setting up of the erection traveller, the first three spans of the viaduct at the Lethbridge end were erected on falsework, by means of a derrick car, and on these the traveller itself was also set up by the derrick car. There were installed on this traveller two 80 h. p. horizontal boilers, and six hoisting engines. Two of the engines were ordinary standard bridge erection six-spool engines, three of them standard 8 in. x 12 in. double-drum engines, and one a special 12 in. x 14 in. engine. This latter was needed to handle column sections of the maximum weight of 33,000 lbs. The large drums on this special engine—using seven lines of ⅝-in. cable and three-sheave blocks—were required on account of the great length of wire cable necessary to lower the column material to the ground. The method of handling the column sections is clearly shown on the drawing of this traveller, and to facilitate the work a number of special shackles were provided.

The clearance under the deck of the traveller is sufficient to permit the passage under it of flat cars loaded. The girders and other material were picked up from the cars by means of four trolleys hung from the traveller on two tracks 8 ft. apart, center to center, these tracks extending the whole length of the cantilever and for about two-thirds of the balancing arm. Each of these trolleys were provided with a 40-ton hydraulic jack with 18-in. stroke, the purpose of these jacks being to raise the material clear of the cars and allow the latter to be withdrawn. If block and tackle had been used it would have been necessary to build the traveller considerably higher, so as to give head-room for the blocks and tackle.

The girders were run out on flat cars in pairs spaced 8 ft. apart, center to center, the same as the trolley tracks, and special girder hooks suspended from the trolleys were secured to the top flanges. The hydraulic jacks were then pumped up, raising the girder an inch or two clear of the cars, and the trolleys loaded with the girders were pulled forward on the cantilever arm of the traveller until in proper position for lowering into place. This lowering into place was accomplished by means of manila rope tackles, provision for their attachment being made in the design of the girder hooks.

As soon as the girders were ready to lower, the hydraulic jacks were released and the weight transferred to the manila rope tackles. The girders were then allowed to spread to their proper spacing, namely, 16 ft. center to center, and were lowered into place. The hydraulic jacks on the trolleys were also used for unloading column material from the cars, the columns being run out and transferred from the trolleys to the wire-rope tackles attached to the bottom chords of the trusses. Lighter material, such as longitudinal and transverse bracing, was unloaded from the cars by wire-rope tackles attached to the trolleys, and were then lowered on these direct to the ground.

For the raising of longitudinal bracing into final position in the work, the traveller was equipped with two 85-ft. swinging booms of five tons capacity each. The transverse bracing was raised into place by wire-rope tackles placed at the end of the cantilever. The floor beams and stringers of the girder spans were unloaded and lowered into place by wire-rope tackles on the trolleys. The manipulation of the trolleys and the operation of the tackles on them was taken care of by a six-spool Beatty engine on the mezzanine platform under the rear end of the traveller. The tackles on the bottom chords of the cantilever for the operation of the boom derricks, erection cages, etc., were provided for by the engine equipment already referred to.

In erecting the bents the traveller was moved into position to bring the hoisting tackles at the end of the cantilever directly over the center line of the bent to be erected, the overhang of the traveller being sufficient to span the maximum space of 100 ft., the length of the intermediate girder spans. Immediately upon the completion of the lower story of a bent supporting 100-ft. span, its top was braced back to the previously completed work by a 100-ft. long temporary strut, shown on the traveller drawing. These struts were put into place by the 85-ft. booms. Similar struts were placed as each story was erected, so that, upon the completion of the bent, it was in a condition to receive a 100-ft. span and to sustain the weight of the traveller when run out to the position required for the erection of the bent necessary to complete the tower. The tower bents were spaced 67 ft. 3 in., center to center.

As the columns for the second bent completing the tower were raised, longitudinal struts and diagonal bracing were put into place. This longitudinal diagonal bracing was made in half lengths, thus requiring splices at the center connection.

To provide for safety and convenience of the workmen in making these connections, two assembling bridges were used, one for each side of the structure. They consisted of lattice trusses, composed of light angles, with a bottom lateral system to give the necessary stiffness, a footwalk for the workmen being obtained by planking over the struts of the bottom laterals. These bridges were put in place by the 85-ft. booms, and temporarily bolted to the cover plates on the columns. This arrangement proved very convenient for the men when bolting up splices in the center of the longitudinal diagonal bracing. In order to give safe access to the upper connections of these longitudinal diagonal braces and the inner ends of longitudinal struts, buckets made of angle-iron frame with wooden sides and floor were used. These were equipped with ladders of sufficient height to enable the workmen to reach all the holes requiring erection bolts. They were also put in place by the 85-ft. booms, and temporarily bolted to the cover plates of the columns.

What is believed to be a novel device, and one which proved very successful and aided materially in expediting the work of erecting towers, was the assembling cage used for the erection of the transverse bracing. This cage, as previously stated, was suspended from the end of the erection traveller, and, as will be seen from the drawing, plate 12, it consisted of a light steel frame containing bottom, intermediate, and upper platforms, at the proper height to give simultaneous access to all the joints of any one story of the transverse bracing. On account of the great width of the bents at the bottom, the lower platform of this cage had to be 111 ft. in length, and it had a height of 47 ft. The cables suspending the cage consisted of two sets of tackles, each made up of seven lines of ⅝-in. steel cable, as it was thought desirable to have a large factor of safety in the supporting tackles to provide for the safety of some 12 to 14 men usually at work on the cage platforms.

The three platforms were connected at their centers by a ladder, and were provided with railings wherever possible. In addition to giving the men safe platforms to work from, the cage was utilized for the important duty of side-hauling the column sections. Ordinarily in viaduct erection, columns are lowered from the traveller and side-hauled to their final position by winches placed on the ground at considerable distances from the center line of the structure. On account of the great height of this structure this method was not considered satisfactory, and, as above stated, tackles of ⅝-in. cable were connected at each

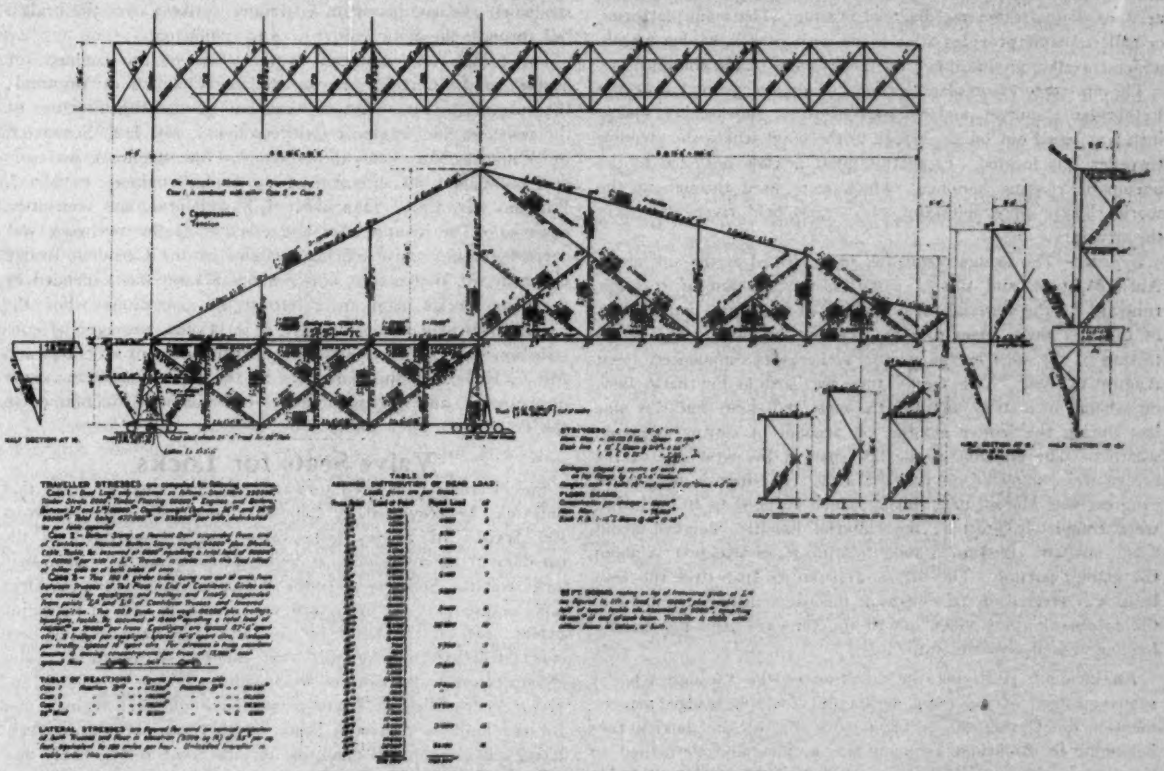


Fig. 6.—Plate 11—Stress Sheet for Main Erection Traveler.

of the extreme ends of the center platform, running to one of the hoisting engines on the main traveller.

For the purpose of signaling to the various engines on the main traveller, a system of telephones was installed, with a telephone booth placed in the middle platform of the erection cage, and each engine operator wore a head-piece holding a receiver and mouthpiece. This method of signaling proved very efficient, the telephone operator in the booth being very close to the workmen engaged in the erecting, and in a position to closely watch all the tackles used for the raising of tower material.

To keep the cage steady in the high winds, the platform (except when the cage was being raised or lowered) was lashed to the columns. It was at first feared that the prevailing high winds at Lethbridge would greatly retard the work of erection, but on account of the unusually safe working conditions provided by the cage, the assembling bridges, and the assembling buckets, the men found no difficulty in working in ordinary high winds, and it was only on days of extreme winds that work had to be suspended.

Riveting Traveller.—To expedite the work of riveting, a traveller was constructed of timber, as shown on the accompanying plate 13, to run on the same track as the erection traveller. It was also built high enough to provide a clearance for the passage of loaded flat cars under it. As will be seen from the inspection of this drawing, riveting platforms and cages were suspended from the overhang of this traveller. The platform or gallery on each side of the tower was suspended at each end by tackle consisting of three-sheave blocks and ⅝-in. cable. To further insure the safety of a large number of men generally at work on these platforms, the cables were wound on worm-geared drums, so that in case of accident to any part of the engine the friction of the worm gear would prevent the free turning of the drums, thus making impossible the fall of the platforms, except from breakage of their supporting cables. It will be noted also that this traveller was made sufficiently long in order that the small cages suspended from each end could be raised or lowered simultaneously on opposite faces of the towers. These small cages were used to provide easy access for the riveting of the center connections of the transverse diagonal bracing. The main platforms or galleries were provided at each end with a truck, having wheels which travelled up and down the cover plates of the columns.

The air compressor plant was placed on the ground near the Lethbridge abutment, and compressed air at 100 lbs. per square inch was piped out on the bridge to the point where the riveting traveller was located. From this point flexible hose led to the pneumatic riveting hammers, which were used throughout the work. There were approximately 328,000 field rivets driven in the structure.

General.—The bridge company's men arrived at the site about April 25, 1908, and started work on the erection of traveller runways for the material yard, building of falsework, and erection of the first three spans and the erection traveller. The actual raising of the steel by the main traveller was commenced about August 15, 1908. Two weeks' time was lost in February, 1909, on account of a strike amongst the men, and some time was also lost during the winter months, on account of stormy and cold weather. The erection of the last span at the west end of the bridge was completed on June 22, 1909. The month of greatest progress was March, 1909, during which bents 37 to 46, inclusive, were completely erected; the material handled weighed about 2,300 tons, and constituted about 735 lin. ft. of structure at about the highest portion. The 167 ft. deck lattice span over the west bank was erected on falsework in the usual manner, timber for the falsework being taken out of the runways used for the unloading traveller in the yard.

An incident, justifying the adoption of the through type of spans (instead of the usual deck type) for this bridge, may be mentioned. During the erection work one of the derrick cars belonging to the bridge company was accidentally overturned on the bridge, but was saved from falling overboard by the side girders. Had it fallen over the side one or more towers would inevitably have been damaged, entailing great loss and delay to the work.

The quantities in this structure are approximately as follows:

Dry excavation	18,110 cubic yards.
Wet excavation	4,870 cubic yards.
Concrete	17,090 cubic yards.
Concrete piling	1,676 piles.
Riprap	339 cubic yards.
Steel	12,200 tons.

It required 645 cars to transport the steel actually used in the erection of the bride to the bridge site. Contractors' plant required about 40 cars, timber ties and guard rail about 53 cars, besides which there were additional cars used on the work for coal handling, which would make the total about 900.

To facilitate the inspection and painting of this structure, a light steel overhead traveller, running on the tops of hte girders, with clearance for traffic, has been designed. The traveller, when constructed, will have fixed booms on either side, with movable trolleys, from which cages will be suspended and operated by a hoisting engine placed on the floor of the traveller over the track. This will provide safe and ready access to all parts of the structure for men engaged in painting.

Throughout the work careful records were kept of all measurements, triangulations, and other work, and, in addition to this, records were kept, as shown on the general plan, plate 1, giving the approximate force and direction of the wind and state of the weather on each day throughout the work. Weekly progress reports of the substructure were made out and sent in to the head office.

During the construction of this bridge four fatalities occurred. Of these, two were due to gas from the old coal workings, to which a shaft had been sunk at pier No. 23 in order to make absolutely sure of the foundations over these workings. A boy had gone into the shaft, notwithstanding orders to the contrary, and two men perished in rescuing him. One man fell from the top of tower No. 47 while making connections on the steelwork, and another man, a stranger, walking over the bridge, fell through the floor before it was completed.

The bridge was designed in the office of the engineer of bridges of the Canadian Pacific Railway Company at Montreal; the substructure was laid out and built under the direction of the assistant chief engineer (western lines), Mr. J. E. Schwitzer, of Winnipeg, Man.; and the erection of the steelwork was carried out, under the direction of the bridge engineer, by Mr. J. Rankin. Mr. C. C. Schneider, of Philadelphia, was consulting engineer. The manufacture and erection of the steelwork was carried out in a most efficient manner by the Canadian Bridge Company ,of Walkerville, Ont. About $75,000 was expended by them in special plant used during the operations. For the Canadian Bridge Company, Mr. A. L. Colby, manager of construction; Mr. J. W. Stoughton, superintendent of erection; and Mr. C. F. Prettie, superintendent on the ground, were in charge of the work, and a gang of about 100 men were maintained in the field.

Valve Seats for Locks

Specifications have been forwarded to the States in order that bids may be requested by the Washington office for the seats and frames of Stoney gate valves to be embedded in the masonry of the twin locks at Pedro Miguel, and the upper twin locks and the Spillway at Gatun. Each of these two twin locks will require two sets of guard valves and 12 sets of operating valves; two of the latter for use at Gatun have already been contracted for, making the total number of operating valve chambers under the present specifications 22, ten for Gatun and 12 for Pedro Miguel. Three frames have also been requisitioned for the Spillway at Gatun Dam, but they are not for the regulating valves that will finally be installed, but for use only during the construction of the Spillway.

The present condition of work on the locks makes it desirable that the castings be delivered by April 1, 1910.

The valves for which these castings will form the chambers are for the main culverts by which water will be let into and drawn from the locks. Each valve is designed to operate in a well, traveling on two roller train bearings with a span of 10 ft. from center to center, fastened to the down-stream face of the wall casting. Each valve closes an opening 8 ft. wide by 18 ft. high. The valves are to be arranged in pairs separated by a pier. The gates will have a total play of only one-fourth of an inch, and the castings must therefore be carefully made and fitted together so that when they are assembled and the valve is in place there will be no interference or bad alignment which would hamper or cripple the operation. The contractor will be required to assemble the castings at some place in the States agreed upon for inspection by an agent of the Commission, and install a template of a complete valve and operate it to the full range of motion without any interference with free and perfect operation.—Canal Record.

The Maintenance of Way Department

Tie Plates and Winter Track Maintenance

Editor, Railway Engineering:—

With regard to maintenance of our winter track. It can all be summed up by saying that the roadbed freezes hard, which necessitates about the average amount of shimming that is generally done in cold weather and a moderate amount of snow bucking.

We have gone into the winter with our track in very good shape and do not anticipate any work on it except a little shimming here and there where track heaves.

Our section crews are much reduced. We allow a foreman and two men on all ordinary sections and our principal work for these men is tightening bolts, tapping down spikes, watching the gage and keeping the switches and station platforms clean.

In laying tie plates where we are putting down new rails we put our tie plates down as we lay the rail, the ties having just previously been adzed so as to give a good seat to the tie plate, and immediately the rail is laid it is spiked, and the spikes driven down as far as the tie plate will allow when it is not sunk into the tie. As soon as a train or two passes over newly laid tie plates we go back over our spikes and tap them down. Even if train service should be such that we could not get a train over some newly laid tie plates by the end of the day's work we would unhesitatingly leave the track as it is during the night with the tie plates not driven into the ties.

On account of the style of our tie plates and method of securing them we do not fear any bad results or consider the track dangerous when left with the plates driven in it without the spikes driven down. Of course it is our aim to tap down the spikes as soon as the plates are driven in by the weight of a train, but if not convenient to do this we don't anticipate trouble.

In laying tie plates where new steel is laid it is very important tant to adze the tie so as to give the plate a good, secure, level seat, and of course it is also important to see that the plate is put on straight so that the spikes and shoulder (if there is one on the plate) fit up securely and squarely against the rail.

In laying tie plates on new ties that we are putting in where tie plates have not been previously used, we sink the plate by means of a sledge into the tie before putting the tie into the track. The object of this is to avoid having loose ties on either side of the ties that have just been put in with the tie plates. I also consider it a better method of getting the tie plates well seated without damaging the tie or plate by driving it in. In laying tie plates on new ties where the plates are put onto the tie before the latter is put in the track, it is very important to have your gauge correct so the plates will fit properly on the rail after the tie is put in.

Yours truly,

Montana. Roadmaster.

Winter Track Maintenance

Editor, Railway Engineering:—

The winter maintenance of track is very similar in all places where the ground is affected by frost. There is very little to be done that is of any benefit to the track after it freezes up.

About all that we pretend to do is to gauge track during pleasant weather when there is not too much snow around the rails, and to keep the bolts tight in the joints. The balance of the time is devoted to shimming and taking care of snow and ice.

We have comparatively little shimming on this Eastern Division, as track is very well ballasted.

Yours truly,

Massachusetts. Roadmaster.

Winter Maintenance and Tie Plates

Editor, Railway Engineering:—

The principal contention in this part of the country during the winter months is the difficulty encountered in maintaining track in surface and line account being badly disturbed, caused from the action of the frost entering the ground, causing track to heave out of surface and line. This necessitates the putting of shims on tops of ties so as to keep track in surface and line. We use regular manufactured shims ranging in thickness from ¼ in. to 4 ins., consisting of bass, ash and some oak wood. The shims are bored by the manufacturer so as to prevent their splitting from spiking them.

Braces are used consisting of hard wood for maintaining rails in position, and in addition we use what is termed a spread rod, which is the same as the old switch rod used for inside switches minus the inside lugs. These rods, in addition to the braces, or even without them, insure the rails from spreading, so that there is no necessity for dwelling upon the value of them.

We also have a good deal of trouble where springs exist alongside of track, being obliged to protect the water from becoming frozen by covering over with brush, otherwise it would necessitate a good deal of labor keeping the ditches open, chopping away of ice, etc., that would accumulate.

The worst feature of heaving is that many miles of the roadbed become affected in a night and, of course, forces being reduced to winter basis, it takes a long time until all the places affected can be shimmed to surface. Speed of trains has to naturally be restricted at all such places until track can be shimmed to surface as above described.

Concerning the laying of tie plates in connection with spring work: This refers to the inserting of renewal ties. We deem it necessary to imbed them into the tie before placing under the rails, which by so doing does not disturb the line or surface of the track, otherwise, by depending upon the traffic to imbed them, the spikes would have to be pulled for some distance on either side of where tie plate was placed, or the track raised a sufficient amount for so doing, which would necessitate its having to be given a general overhauling, which, to say the least, would not be a very economical or commendable method of handling this class of work.

If there is anything further that I can tell you of interest, will consider it a favor to be called upon.

Yours truly,

Wisconsin. Roadmaster.

Winter Track Maintenance

Editor, Railway Engineering:—

Our forces during winter are reduced to just sufficient men to keep track smoothed up and in line and surface. We attempt no other work when track is not frozen too hard to work upon. In event it is frozen too hard, we use the men in cleaning up around station grounds and other such work that can be done to an advantage. In event of much snow, men are all used to keep switches, interlocking plants and platforms free from snow and ice. If conditions are such that no work can be done to an advantage, men are all laid off and the foreman walks track.

Yours truly,

Kentucky. Roadmaster.

Tie Plates and Winter Track Maintenance

Editor, Railway Engineering:—

This season of the year in this section of the country we are not bothered with track heaving and do not have to resort to shimming as they do farther north. We can carry our work on through the winter, with the exception of a few days occasionally, as we do through the remainder of the season.

Our winter track work consists of tightening bolts and gauging, spotting up low spots as they develop in our ballasted track, using the tamping bar or pick, cleaning right-of-way, repairing fences, which can be done as well at this season of the year as any other time.

In regard to applying tie plates, of course the best methods of applying tie plates is to put them on at the time the tie is put in; but to obtain the best results in applying them out of the face on ties if they have been laid is to, first adz a smooth face on the tie by pulling the spikes, plug the spike holes, apply the plates for several panels on the line side, maintaining the gauge and taking care of the traffic, then repeat the operation on the gauge side. This work can be conducted without any interference to traffic.

I am a great believer in the use of tie plates. Experience has taught the users of tie plates that when they first began to make them that they were made entirely too light, but the styles and patterns they are making nowadays seem to be about the right thing.

Yours truly,

Oklahoma. Roadmaster.

The Substructure of the Passyunk Bridge*

The bridge now in process of construction across the Schuylkill River on line of Passyunk Avenue at Point Breeze, Philadelphia, consists of twelve spans of steel superstructure on concrete masonry (Fig. 1). The eight spans of the west approach are 100 ft. each, c. to c. of bearings, of alternate continuous and cantilevered plate girder deck construction; the two spans flanking the channel are 125.4 ft. each, of riveted lattice deck trusses; the channel span is a double leaf trunnion bascule, 223 ft. between piers, with riveted trusses. The east approach span is 47 ft., the portion immediately over and close to the surface of the bank being of steel I beams incased in concrete. This portion is carried at its outer end on a 25 ft. cantilever from the lattice flank span. The total length of the bridge is thus 1,323 ft. The deck will be 56 ft. wide with 36 ft. driveway paved with asphalt and two 10 ft. sidewalks paved with granolithic, except on the bascule, where both will be paved with wood.

The clear width of the channel between the fenders is 200 ft., and with the two bascule leaves raised 80 degrees—the trunnions being 16 ft. back from the fender line—this clear width will be maintained to a point 135 ft. above high-water line. The slight projection of the leaves beyond the fender line above that height will not foul with the masts of the largest vessels. The elevation of the floor of the bridge was fixed at a height that will give a clearance over mean high-water line when closed of 33 ft. under the floor beams and 35 ft. under the stringers between the floor beams—sufficient for the passage of ordinary tugs, and reducing to a minimum the necessity for opening the draw.

The west abutment and nine of the piers are of rubble concrete on wooden pile foundations. Piers 9 and 10, on either side of the channel, are also of rubble concrete; but were carried down to and founded on bed rock by pneumatic process in wooden caissonse. The east abutment is merely a shallow concrete pier founded in the gravel stratum of the east bank of the river, the slope being spanned by the superstructure, a design which was found to be more economical than that of a retaining abutment at the site of pier 11.

The two channel piers are protected by framed and braced pile fenders with independent clusters of piles (Fig. 2).

The substructure work has been completed and has been done at different times after successive appropriations by Councils and under three separate contracts. The west abutment and Piers 1, 2, 3, and 4 were constructed by David Peoples in 1903. Pierce 9 and 10 were constructed in 1908 under contract of the Filbert Paving and Construction Co., and Piers 5, 6, 7, 8, and 11 in 1909 by McGaw & Gray.

The east approach to the bridge is on high ground, the present grade of Passyunk Avenue requiring to be raised only about 4 ft. at the end of the bridge. The west approach is over low swampy land, for a considerable distance requiring a fill of about 27 ft. at the abutment. The surface is in general at about ordinary high-water line, and therefore covered by unusual high tides and freshets. The soil is a stiff marsh mud to a depth of perhaps 30 ft., and so yielding that a bank of earth placed on it will sink into it and squash the mud up at the sides. The foundations of the west abutment and piers required, therefore, to be braced to resist horizontal forces, because the filling behind the abutment will tend to thrust the abutment forward, and if filling should be done in the future on only one side of the approach piers, a lateral thrust will be developed. This bracing was accomplished by driving spur or batter piles on all sides of each foundation.

The pier in the slope of the east bank—No. 11—has sufficient embedment in a clay and gravel bank to secure it firmly against lateral displacement. But it is subject to some longitudinal thrust from the bank tending to tip it toward the river. Therefore all the spur piles that could be driven in this foundation are in the one direction—riverward. To lessen the thrust the slope immediately back of the pier was excavated to the top of the foundation, and the new slope made 45 degrees and paved with mortared rip-rap to hold it.

The design of the west abutment is interesting. It was made by Mr. C. M. Mills, member of the Club, while he was in charge of the design and construction of bridges in the Bureau of Surveys. It is one of the early examples of the use of reinforced concrete. The front of the abutment is extended at the base about 9 ft. for the purpose of securing a uniform distribution of load over the piles. The center of pressure resultant from the weight of the masonry, the reaction load of the steel superstructure, and the thrust of the earth filling back of the abutment, as figured by Rankine's formula, falls at the middle of the base, as figured by Rankine's formula, falls at the midde of the base, and therefore gives as nearly as practicable the same vertical load to each of the plumb piles. This load is 16 tons per pile. The spur piles are not regarded as carrying vertical loads. To make the extended toe competent to safely carry the distributed

*From a paper by Henry H. Quimby in Proceedings of Engineers' Club of Philadelphia.

load it was reinforced as an inverted cantilever with 1 in. square steel rods 12 in. c. to c., embedded 6 in. above the pile caps. Fig. 3 shows all the dimensions and the filling that was done under the contract. The three pilasters carry the seats for the three girders of the superstructure, and although constructed monolithically, they are tied into the body with steel bars. The pile foundation was extended at the back 31 ft. and covered with two layers of 6-inch yellow pine planks to constitute a platform to maintain the filling immediately back of the abutment. This filling of earth and gravel was placed as a means of securing the conditions assumed in the calculations of the stability of the abutment, because the filling of the approach will be done perhaps under other supervision in the future, and the material to be used in filling cannot yet be known.

The piles used throughout were long-leaf yellow pine, not less than 8 in. at the point and not less than 12 in. at the butt when cut off, which was done at eevation—10, mean low-water being—8.5. The plumb piles used in the foundations west of the channel were from 50 to 56 ft. long and the spur piles 68 to 72 ft. Under Pier 11, 26 ft. plumb piles and 30 ft. spur piles were driven. The spacing is from 3 ft. 9 in. to 4 ft. on centers. The plumb piles are capped with 12 in. by 12 in. yellow pine, and the spur piles generally check against 12 in. by 12 in. stringers under the caps, though some that were found to be out of line were not strained into line, but left for mere embedment in the concrete.

All foundation piles were driven with a steam-hammer; those of the fenders were driven with the ordinary drop. The steam-hammer used in the first contract was the heaviest of the Vulcan type, weighing over 9,000 lbs. in all, the piston and ram being about 6,000 lbs. The stroke is about 36 in., and with 60 lb. steam pressure it strikes about thirty blows per minute. The cylinder is single-acting—merely raising the piston and letting it drop of its own weight. The same form of hammer, but only two-thirds as heavy, was used in the last contract. Considerably longer time was required to drive a pile with the lighter hammer, many hundreds of blows being required, though almost no brooming or splitting resulted. The heaviest steam-hammer is probably the most economical to use.

The slow penetration of the piles under the lighter hammer—often only one thirty-second inch per blow for several feet—suggested the possibility of the points getting broomed. Accordingly a number of the piles were shod with iron points or shoes—nineteen in all were tried—but no difference in the driving could be perceived, and it was evident that the resistance to driving was in the skin friction and not at the point.

The spur piles were intended to be inclined at an angle of 30 degrees with the vertical, but it was difficult to secure so much except at Pier 11, when the leads of the driver were suspended from a derrick boom. All others were driven by inclining the pile driver backward at an angle of 15 degrees and setting the pile at a still greater angle, driving it a short distance, then releasing it, whereby it would spring up, then moving the driver up to it and driving it some more, continuing this action until the pile was down to practical refusal.

The mud is of such a depth that the piles sank into it under their own weight and that of the hammer resting on it as much as 30 ft. below the bottom of the excavation before a blow was struck.

All piles were surrounded with concrete to a depth of 4 ft. below the cut-off. This was for the purpose of stiffening the piles which are for perhaps 30 ft. of their length in a medium that will afford very little resistance to buckling under the compression as columns, and the fixing of the top ends in concrete with the fixing of the points in 10 or 15 ft. embedment in gravel, gives to each pile two points of contra-flexure.

To maintain the integrity of the concrete and secure this fixing value for the outer piles steel bar loops were embedded in the concrete around them near its lower surface extending back past the second row of piles. These should prevent any cracking of the concrete on line of the outer piles.

The piles were barked where the concrete was placed around them. To secure it to them and prevent slipping heavy spikes were driven into the wood. The mud has very little sustaining value—probably little more than that of a fluid of about 100 lbs. weight per cubic foot with a slight head. Very little water permeated the mud, but that little mixed with it under the feet of the workmen, making a black liquid that penetrated and "killed" any concrete that might be deposited on it, and it was necessary to lay boards on the bottom, fitted between the piles, before depositing concrete.

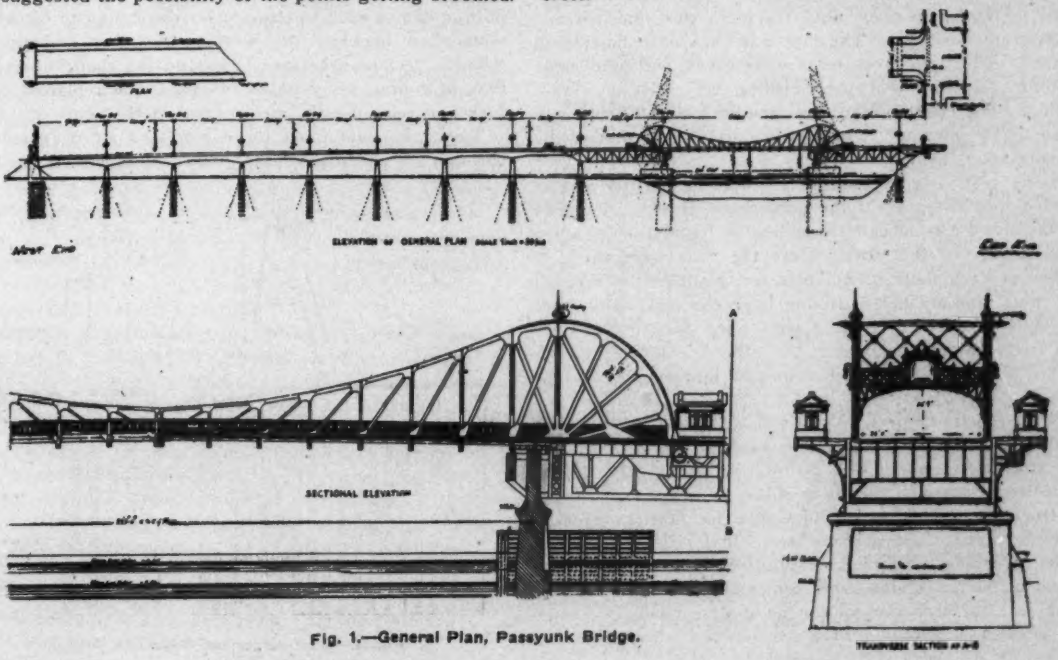


Fig. 1.—General Plan, Passyunk Bridge.

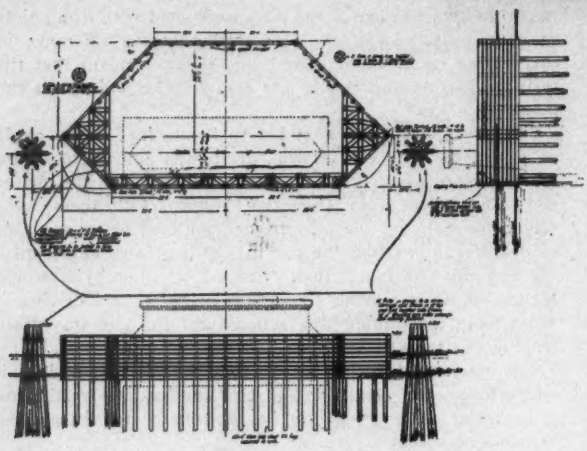


Fig. 2.—Fender, Passyunk Bridge.

The excavation of the mud to plan depth was done in each case before the piles were driven. After driving it was always found that the bottom of the excavation had come up among the piles—sometimes as much as 2 or 3 ft.—and had to be re-excavated. The excavations for the piers were all sheeted and very heavy waling and rangers were required, for the mud is mobile and lies very heavy against shoring. For the west abutment foundation the hole was not sheeted and the sides were sloped in. It was large enough to manoeuvre the pile-driver on the bottom of the hole, but the pier holes were not, and they required heavy timbers to span them for carrying the driver.

All concete is of the hybrid or rubble character—crushed stone concrete, 1:3:6, with large stones embedded in it by hand. All cement was Portland. The showing surfaces were made with granolithic mortar, a fine concrete, 1:2:3, placed against the forms. In the case of piers constructed in warm weather when the setting of the cement was normal, the face forms were removed on the next day after depositing and the surface scrubbed with brushes and water sufficiently to remove the film of cement and expose the sand and stone of the fine concrete. The piers of the last contract were constructed in cool weather when the setting of cement was uncertain and irregular, wherefore the face forms were permitted to remain in place until the piers were finished and the concrete was hard. The face was then bush hammered by hand. The two river piers were faced with the same granolithic mixture, but the finishing was done by sand-blasting. The effect of the three methods of surfacing will be practically the same in appearance after the action of the elements upon them for a time.

All the piers have embedded in them at bottom and at top steel rods to prevent the occurrence of vertical cracks that might arise from partial yielding of foundation or some other cause. In the earlier piers the rods were three in number at each point, 2 in. diameter, plain, with threaded ends, nuts, and washers. In the later contracts they were 1 in. square formed or twisted reinforcing bars, in the river piers twelve in number and in the other piers nine at each point. No cracks have yet appeared in any part of the work.

The two channel piers, Nos. 9 and 10, were constructed in wooden caissons sunk by compressed air process to bed rock at elevation about —88.

The fenders around the three sides—ends and back—were constructed first and used as platforms for derricks, storage of lumber, etc. The caissons were built in position suspended over the exact pier location. Temporary piles were driven in four pairs along the front and the rear of the pier so as just to clear the caisson as it descended. On each row of piles was laid a 12 in. by 12 in. yellow pine cap parallel with the caisson. Underneath these caps and between the piles of each pair transverse 12 in. by 12 in. yellow pine beams were suspended just high enough to clear high tide, by means of 2 in. screw-rods about 16 ft. long, threaded the whole length. There were two rods to each end of each beam—sixteen rods in all. These rods were provided for the purpose of lowering the caisson into the water when ready. They passed through the caps, the upper or working nuts bearing on the caps, and carried at their lower ends iron straps under the cross-beams (Fig. 6).

The details of the caissons are shown in Fig. 5. The cutting-edge consisted of a 24 in. by ½ in. steel plate with a 6 in. by 6 in. steel angle riveted along its lower edge, back up. On the angle was placed the first layer of 12 in. by 12 in. yellow pine. On this and inside of the steel plate the siding was placed. It consisted of 6 in. thick dressed yellow pine planks in vertical position, grooved both edges for 2 in. by 3 in. spruce splines, which were driven into place from the top end of the planks of each story, and the joints were caulked on outside and inside with oakum. The walls of the caisson or working chamber were made of layers of two 12 in. by 12 in. yellow pine, drift bolted together, and strutted across with three 12 in. by 12 in. timbers dovetailed in and drift bolted. The ceiling of 3 in. yellow pine planks was supported by posts and cross-beams, without spiking, in order to be easily removable before concreting the chamber, the design being to secure a pier of continuous concrete without any interposed layer of wood. The wood ceiling was therefore only a temporary casing, serving to make the working chamber air-tight through the caulking of its joints, and to carry the first layer of concrete, which was reinforced with 1 in. rods, spaced 12 in., laid transversely. This reinforced layer furnished adequate support for the succeeding layers, which were all bonded together with the rubble embedded stone.

Three circular well holes, 3 ft. 6 in. diameter, were framed in the ceiling, and collapsible segmental cast-iron shafts connected to them, the two end ones for hoisting out excavated material and the middle one for entrance and exit of the workmen. These shafts were carried up in story lengths to sufficient height to be above water upon first sinking, and as excavation proceeded and the caisson was lowered, additional stories were put on. The air-locks were at the tops of the shafts, but a wooden flap valve was fitted at the bottom end of each to close under the ceiling to cut off communication between the working chamber and the shaft when it became necessary to extend the shaft, which operation, of course, necessitated removing and replacing the air-lock and releasing the air-pressure in the shaft.

Two 5 in. inlet pipes for compressed air were connected through the ceiling at one corner and provided with flap

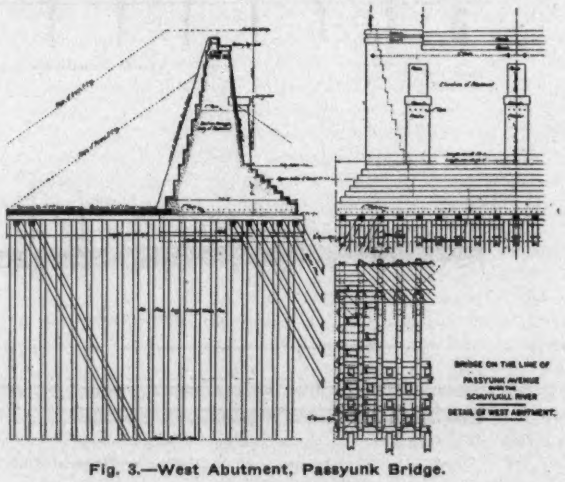


Fig. 3.—West Abutment, Passyunk Bridge.

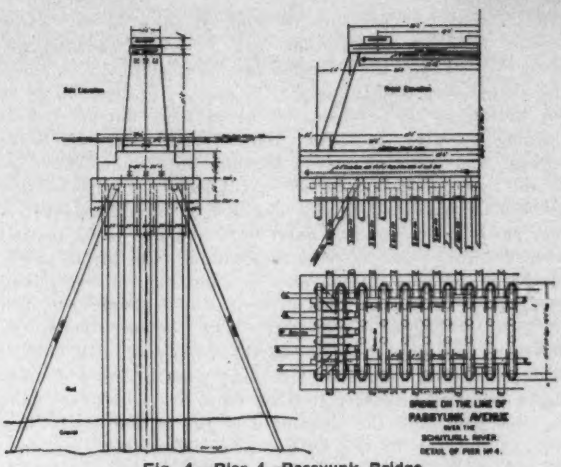


Fig. 4.—Pier 4—Passyunk Bridge.

valves which would check the escape of the compressed air in the chamber if an accidental break of a supply pipe should occur, as once actually did happen to the pipe laid across the river on the bottom, a sinking barge having lodged on it. A 4-in. pipe was carried up near the middle and fitted with a screw valve to reduce the air-pressure when desired or to blow out water and excavated material. A small pipe was carried up outside each shaft and provided with a valve in the working chamber and a whistle at the top end for signaling purposes. Wires for incandescent electric lights were also run into the chamber. All these pipes and wires were surrounded with the concrete of the pier and left in upon completion.

When all these connections were complete and the sides built up to a height of about 30 ft., the structure, which weighed about 200 tons, was lowered into the water by simultaneous unscrewing of the sixteen suspender rods. With all its valves closed it was a water-tight bottom and floated when sufficiently immersed. It was carried on down by depositing the concrete in it, 1 ft. of concrete sinking it about 2½ ft. (Fig. 7).

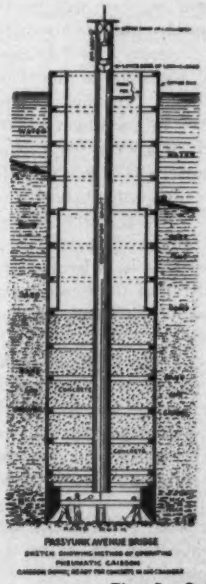
When the cutting-edge entered the river bed air was forced through the pipes into the working chamber and the water thus driven out below. Then the compressed air workmen —called "sand hogs"—went into the chamber and commenced excavation.

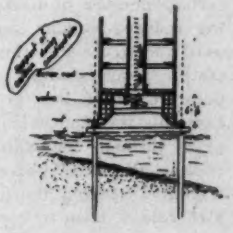
The spoil is sometimes of a nature that will permit of removal by means of a centrifugal pump, or even by merely letting the compressed air blow it through the pipe. For such disposal the spoil must be mixed with a considerable volume of water. The water is obtained by simply keeping the air at a pressure that will permit just enough water to enter the sump into which the spoil is gathered and out of which the water is being blown. If the spoil is of a character that cannot be carried by the air and water through the pipe, it must be shoveled into buckets and hoisted out through the shafts and air-locks.

The ejectment of water and sand through a pipe standing considerably higher than a static column of water that would balance the actuating air-pressure is accomplished by permitting air to mix with the outgoing fluid—the strainer inlet of the blow-pipe is not completely submerged in the water that is being driven into it. If the surface of the water in the sump is 40 ft. below the surface of the river outside of the caisson, a pressure of air of 17½ lbs. per sq. in. will prevent more water coming through the bed, but that pressure will drive the mixture of air, water, and sand to a height of perhaps 50 ft. to discharge over the side of the caisson. This is due to the fact that the admixture of air with the water lightens the cubic weight of the contents of the pipe to less then that of water alone.

The spoil buckets are locked into and out of the compressed air very quickly by means of spherical valve gates so shaped that the internal pressure holds them tightly shut when shut, but does not offer much resistance to the opening, which is done by a balanced lever. The upper or outer gates are notched to fit around a stuffing box through which the single hoisting cable slides when the valve is closed on it, and which remains at the same point on the cable when the valve is open and the buckets being swung around to the dump. Both lower and upper gates are edged with soft-rubber gaskets, which need to be watched and kept in order to prevent serious loss of air by blowing out when working at high pressures.

The man lock is much longer than the material lock because it is desirable and economical to have it large enough to take a whole shift of workmen at once. The valves are flat, swinging up under the cover and under the floor of the lock chamber, which is an iron cylinder 3 ft. 6 in. diameter, with two iron ladders to which the men cling during the time occupied in taking in or releasing the air-pressure. When the lock is open to the outer air the lower valve is, of course, closed, and it is held by the air-pressure below it. The passage for the men is a circular hole about 22 in. diameter. The valve is a steel plate, perhaps ½ in. thick, weighing about 60 lbs. As the area of the opening at the rubber gasket is about 400 sq. in., less than ¼ lb. per sq. in. difference in pressure of air will hold it shut. In entering the lock, as the last man in lifts the flap the lock-tender—whose duty it is to be at the air-valve constantly—lets in the pressure from the shaft below through a 1 in. pipe, and instantly the effect is felt throughout the lock, because of the extreme homogeneity of air. The pressure at once grips the flap valve, which has not any other fastening. As soon as the pressure in the lock is the same as that in the shaft and working chamber below, the lower flap drops and the men climb down the ladder. Consideration for the safety of the men in the chamber requires that when the man lock is not in operation it shall be kept open to the shaft below to provide a quick refuge for the men in an emergency. When locking out, the air is released slowly, the outlet air-pipe being smaller than the inlet. As the increasing depth of the working requires higher air-pressure the outlet from the man lock is reduced by screwing on to it reducing fittings. At the highest pressure used on this work—38 lbs.





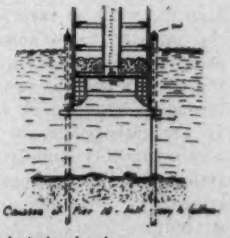


Fig. 5.—Caisson with Bucket in Lock.
Fig. 6.—Caisson Support During Construction.
Fig. 7.—Caisson Partly Sunk and Beginning to Float.

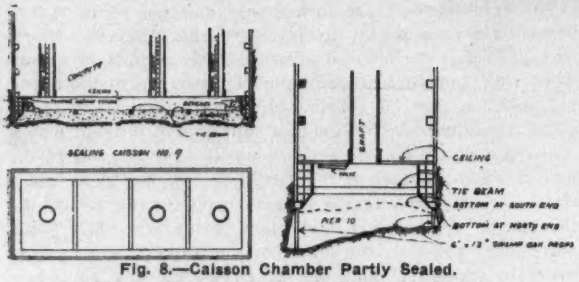


Fig. 8.—Caisson Chamber Partly Sealed.
Fig. 9.—Chamber of Pier 10 Ready for Sealing.

per sq. in.—the size of the outlet was about 7-16 in., and about twenty minutes time was required for the lowering of the pressure to that of the outer air. The slow reduction of pressure was for the purpose of preventing caisson disease, called "the bends."

The theory of this disease, which in its more common form causes stiffness and pain in the joints, and in its severe form causes pain in the abdomen and stops the circulation of the blood, resulting in unconsciousness and death, is that long-continued immersion in compressed air causes small bubbles of air to penetrate the tissues and the blood-vessels of the body, and if the pressure be released more quickly than the bubbles can escape, they expand in and congest the minute blood-vessels and hinder the circulation, producing pain similar to that of rheumatism.

On this work there were many cases of "bends," but no lives were lost. In the light of knowledge gained from recent operations of this nature the specifications for this work required the contractor to instal and maintain in constant readiness for use a hospital air-lock for immediate treatment of possible cases of caisson disease, and it is probable that at least one life was saved by the use of it. When a man suffers from the bends after coming out of the air, a restoration of the pressure upon him relieves him almost instantly. In the hospital lock as long a time as may be necessary can be devoted to the gradual reduction of the pressure. It is merely an iron cylinder about 7 ft. in diameter and 11 ft. long, divided into two small compartments, the inner having two beds in it, and both being fitted with pipes, valves and gages.

The compression of air condenses the heat in it and increases the temperature, for if the free air contains a certain number of heat units, the same number will be in the reduced volume, with proportionately higher intensity until radiated. For this reason it is necessary to refrigerate the compressed air supplied to the workings, or the temperature would be too high for the men. In the present case this was conveniently accomplished by passing the air-pipes through the water of the river, but even with that cooling the atmosphere in the caissons was always quite warm. Lowering the pressure lowers the temperature and also causes mist to form. In the air-lock as soon as the outlet valve is opened the chill of falling temperature is felt and the air becomes foggy, the moisture in it being condensed by the drop. If the air be permitted to escape very fast ice can be observed forming around the edge of the outlet pipe in a few seconds.

In the work of excavation care must be exercised to dress the sides so as to keep the caisson plumb and level as it descends. Elevations are taken every day on the top at all the corners and the results given to the superintendent, who is guided by them in directing the excavation and sinking. In the early stages of its descent it can be "thrown" one way or another slightly by inclining the sides of the excavation below the cutting-edge, but after deep embedment this control is very difficult, so it is important to start the descent as nearly perfectly plumb as possible.

Care must also be taken to adjust the air-pressure to the depth of immersion, for considerations for both economy and safety.. Sufficient pressure is required to keep the water down. In a porous stratum such as gravel or sand the water will stand at the elevation that hydrostatically balances the air-pressure, and it vanishes or appears in depressions of the bottom as the intensity of the pressure fluctuates with opening of valves or leaks. Excessive pressure will cause leakage and waste of air by blowing out around the edge of the caisson, or it may lift or tilt and upset the caisson, as actually happened in this city a few years ago, when a pier partly concreted was overturned and had to be blasted to pieces to be removed from the channel. The lifting power of the air under a caisson may be enormous. Every pound of pressure per square inch of surface will sustain a weight of 1 ft. in height of concrete. The pressure which the Passyunk workings reached—38 lbs. per sq. in.—will float a body of concrete above it 38 ft. deep, and therefore in sinking it becomes necessary to place concrete in sufficient body to overcome both the sustaining power of the compressed air and the skin friction of the sides. Sometimes this friction is so great that the weight of the concrete is not sufficient, and hundreds of tons of cast-iron weighs or railroad rails are piled on the pier to force it down. Some foundation engineers systematically "blow" it, i. e., reduce the air-pressure temporarily to withdraw the support of the air, and thus let the weight of the pier act to overcome the friction. This method was used in sinking piers 9 and 10. The last drop of No. 9 was about 3 ft. 3 ins. Bed-rock was reached and the irregularities chiseled off to obtain uniform clearance of 39 in. plumb under the cutting-edge all around. After careful inspection of every foot of the hewn sides of the excavation to make sure that no protuberances of rock should be left to catch and jam the cutting-edge, all hands left the working chamber. Then all outlet valves were opened and the air-pressure, as indicated by a gage outside, fell gradually from 38 lbs., which was carrying 4,300 tons of the pier, to 19 lbs. per sq. in., when suddenly the pier dropped 28 in., fetching up on the cushion of the suddenly recompressed air. which ran up at once to 41 lbs. per sq. in. This was due to the reduction of the volume of the air in the chamber, which was piled more than half full of the spoil from the excavation of the trench under the cutting-edge. During the trenching the spoil was not hoisted out, for fear that the opening of the bucket locks would, by jarring the pressure, start the caisson dropping before everything was ready for it and while the men were in it and exposed to the danger of the sudden increase of pressure, which might rupture their ear drums. The descending pier, which at this time was of about 7,000 tons weight, was thus checked before it was all the way down, and the valves were left open until the pressure again fell sufficiently to let the pier drop the remaining 11 ins. This time it fetched up hard against the rock bottom of the trench and was permitted to remain there—82.3 ft. below city datum.

The next operation was removing the spoil and cleaning off the surface of the rock preparatory to filling the chamber with concrete. Holes were drilled 5 ft. deep in the rock to make sure that it was solid. Numerous leaks of air existed around the edge, and to stop these dry Portland cement was held in such a position at each that the escaping air carried the cement up outside the edge. The cement would adhere to the damp sides and gradually close the opening. Then concrete mixed pretty dry was tamped into the trench against and under the flat of the 6 in. edge angle and wet concrete was deposited over the whole surface of the rock. Then benches of concrete about 2 ft. high and from 2 ft. to 3 ft. wide were constructed around the sides and ends of the chamber, gradually stepping up to the ceiling, the wood ceiling being all removed. This benching was accomplished by using concrete of stiff consistence with one-man-size stone embedded, the faces of the steps being constructed with the larger stones laid up as bulkheads, using the concrete for

mortar. Each tier of the benches was built to within about 2 in. of the ceiling, and after it had set 2 in. space was tightly tamped up with a stiff mortar of cement and sand. (See Fig. 8.)

The successive tiers of benches gradually lessened the size of the working chamber until it became only a narrow gallery about 3 ft. wide and 4 ft. deep connecting the air-shafts. This space was filled by dumping wet concrete into the material shafts clear from the air-locks, and the completeness of the filling was indicated by the flow of the viscous fluid to and up into the man shaft. The air-pressure was maintained all this time through pipes entering the shafts for the double purpose of keeping the water down and of forcing grout from the concrete into any crevices that might form by the shrinkage of the body of concrete in the chamber, and therefore every batch of concrete required to be locked into and out of the shaft, one of the "sand-hog" gang foremen going into and out of the lock with every batch to dump the bucket, alternating from full pressure to no pressure every two or three minutes, without appearing to suffer any ill effects.

When the concrete filled about 8 ft. up in the shafts the sealing was regarded as complete and concreting was stopped, but the air-pressure was maintained to keep the ground water from washing in through the concrete before it could set hard. The lower lengths of the metal linings of the shafts were thus necessarily left in the pier, but all the upper lengths were of collapsible pattern, to facilitate removal before filling up the shafts. This work required the air-pressure to accomplish until all lengths were removed to above the water-line. Then the locks were removed and the water finally found its way through the concrete into the shafts, and the filling of the shafts had then to be done in the water. Automatic cylindrical dump buckets were used, so fitted that when filled and swinging on the derrick-hook the weight was carried on inside chains fast to the bottom flaps of the bucket, but on landing the bucket two hooks on a sliding guide bar attached to the chains would drop by gravity and automatically engage the bail of the bucket. Then when the derrick-hook was raised the bucket would rise and the bottom flaps would open, letting the concrete flow out below, thus depositing the concrete with a minimum of agitation of the water.

The concrete plant was on a scow which was moved from one pier to the other as desired, the exigencies of sinking the caissons causing irregular work of the plant. As the pneumatic work proceeded day and night, two gangs of concreters were required, one for night work, and the shafts of inspectors were arranged so as to provide constant supervision.

The rock bottom under Pier 10 is far more irregular than that under No. 9. The southeast corner struck it first, and it was so hard that blasting with small charges—1 to 2 lbs. of dynamite—was resorted to to make a ditch there for the cutting-edge and to bench off the sloping surface into ledges for satisfactory bearing. The caisson was not sunk any lower, but remained at elevation—82.5 while excavation at the north end continued down to—87.4, the surface of the rock sloping in that direction irregularly. The north end of the pier was thus held up only by the compressed air and the skin friction, and to secure it against dropping there and tilting, through such a possible accident as loss of air-pressure before the concrete support could be placed, a number of 6 in. by 12 in. swamp oak posts were inserted under the hanging edge and wedged tightly between it and the rock surface. The excavation at this end was carried out beyond the outside of the caisson 12 in. to increase the width of base of the pier as some compensation for the increased height. (See Fig. 9.)

The material of the stratum immediately overlying the rock is a mixture of micaceous rotten rock and white clay or decomposed feldspar, and could generally be removed with shovels without picking. The stratum is perhaps 20 ft. thick and is at approximately the same elevation on both sides of the river.

The intensity of the load on the foundations of these river piers depends upon the penetrability of the bottom to water. If the water can permeate the bed and act against the under side of the concrete the weight of the pier will be very materially lightened by the flotative property of the water, every cubic foot of the pier below the surface being thus lighter by 62½ lbs .than it will be if the water cannot act against the bottom. In the one case the maximum pressure on the foundation will be 6 tons per sq. ft. and in the other case 4 tons. This assumes a load central with the pier. In the event of wind blowing against the raised leaf of the bascule superstructure there will be an overturning moment on the pier. Although the superstructure of the flank spans will rigidly connect each channel pier with the adjacent pile pier, we may assume that the overturning force is wholly resisted by the river pier. As the surface of a compact sand and gravel stratum is not more than 54 ft. below the point of application of the wind-force—the bottom of the steel work—the lever arm of the moment may be taken at 56 ft. Assuming a wind-pressure of 15 lbs. per sq. ft. of raised bridge floor, which is far beyond what would permit the bridge to be lifted, but might be reached by a gust striking it while up, the overturning force will be 90,000 lbs. and the center of resultant pressure will be only 9 in. from the center of the pier. Even if the embedment of the pier in the firm material that embraces it for 50 ft. of its height were to be disregarded, the center of the resultant at the rock bottom would be only 6 in. and 9 in. respectively for the full weight and the flotation assumptions, increasing the pressure at the edge of the base on the rock to 6.4 tons and 4.6 tons re-

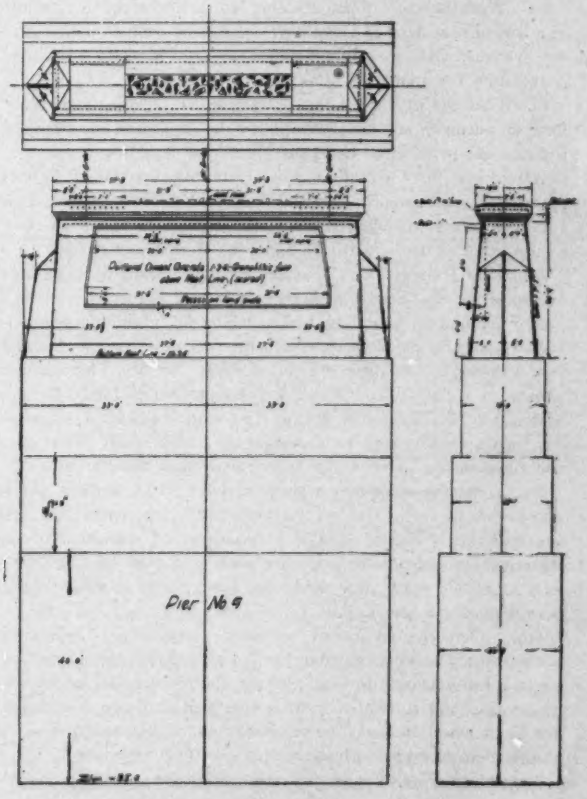


Fig. 10.—Design of River Piers, Passyunk Bridge.

spectively. The maximum possible therefore is less than 100 lbs. per sq. in., which is a very low stress for even wood.

The bed of the river on the east side has no deposit of mud, a the west side has. This is probably due to the action of the current in the channel, the east side at this point being the apex of a sharp bend in the river, and such conditions are liable to cause scouring of the sandy bottom after the planting of obstructions such as the fender and the pier. No provisions have yet been made to prevent scour, either around Pier 10 or on the bank in front of Pier 11, but soundings have been made and will be made at intervals to ascertain whether scour does occur, and if it should develop the vulnerable points will be protected with rip-rap.

The tota cost of the substructure work was $261,190, of which $53,060 was for the west abutment and Piers 1, 2, 3, and 4, $158,600 for Piers 9 and 10 with the fenders, and $49,530 for Piers 5, 6, 7, 8, and 11.

The contract for the superstructure has been awarded to the Strobel Steel Construction Co. of Chicago at the lump sum price of $313,000 for the work complete, including paving and lamp-posts, but no grading of approaches. The east approach is provided for in the same contract to be done under unit price items, and the estimated quantities will make it cost $8,000 additional. This east approach work, which includes street paving and repaving of sidewalks, will make the bridge accessible for use to the west abutment, and will thus afford means of hauling material for grading the west approach from either side of the river and expedite the completion of the highway and open it to public use.

The Signaling Department

Railway Signal Standards

In presenting the present series of articles on Signal Standards, it has been thought proper to place first the standards of the Railway Signal Association relating to automatic block signaling in so far as the subject has been acted on by the association. These standards are presented herewith in part only. Selection has been made of parts of the various specifications, only those of a descriptive character being given; everything relating to contracts, tests and similar subjects being omitted as not consistent with the object of the articles. Some of the matter has not as yet been approved officially by the association, but forms the subject of reports of the various committees and has by them been recommended to the association for adoption.

In connection with Figs. 1 to 6 inclusive, published on pages 196 to 199 in the October, 1909, issue of Railway Engineering, the Committee No. 1 of the Railway Signal Association recommended the adoption of the following conclusions:

1st. That the indications, Exhibit No. 1, are adequate; permit of a uniform system of signaling; are not in conflict with existing systems, and are recommended to the American Railway Association for approval.

2d. That the primary aspects, Exhibit No. 2, are practicable; form an adequate and proper basis for the display of the Primary Indications; provide an excellent means for attaining a uniform, universal system of signaling; and are therefore endorsed by this association, and submitted to the American Railway Association for such action as may be necessary to enable roads desiring to use them to do so with the approval of that association.

3d. That Exhibit No. 4 provides a simple means of signaling; adequate for the needs of many roads and branches, and is a proper method of signaling which, while not providing for uniformity, may be developed through the scheme shown on Exhibit No. 3 into the complete system, Exhibit No. 2. The outlines shown on Exhibits Nos. 3 and 4 are therefore endorsed by this association and submitted to the American Railway Association for such action as may be necessary to enable roads desiring to use them to do so with the approval of that association.

4th. That the secondary aspects, Exhibit No. 5, form a simple and proper means of conveying certain necessary instructions, and are therefore endorsed by this association and submitted to the American Railway Association for such action as may be necessary to enable roads desiring to use them to do so with the approval of that association.

5th. That the additional secondary aspects in combination with primary aspects, Exhibit No. 6, are suitable and proper aspects for use where desired, and are therefore endorsed by this association and submitted to the American Railway Association for such action as may be necessary to enable roads desiring to use them to do so with the approval of that association.

Regarding Figs. 9 and 10, the committee reported as follows:

Two designs of semaphore spectacles were found necessary, because the approved scheme of aspects requires a staggered light on automatic signals; i. e., the "Stop and then Proceed" indication; and as the stagger has in every exhibit of aspects to date shown the top light to the right and the second light to the left

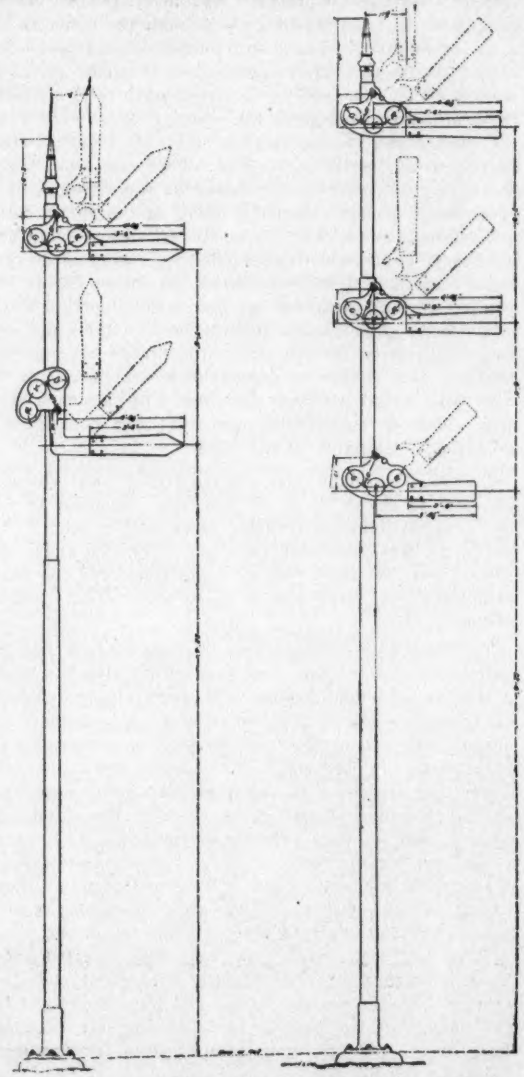


Fig. 7.—Two Arm and Three Arm Signal Complete. Railway Signal Association.

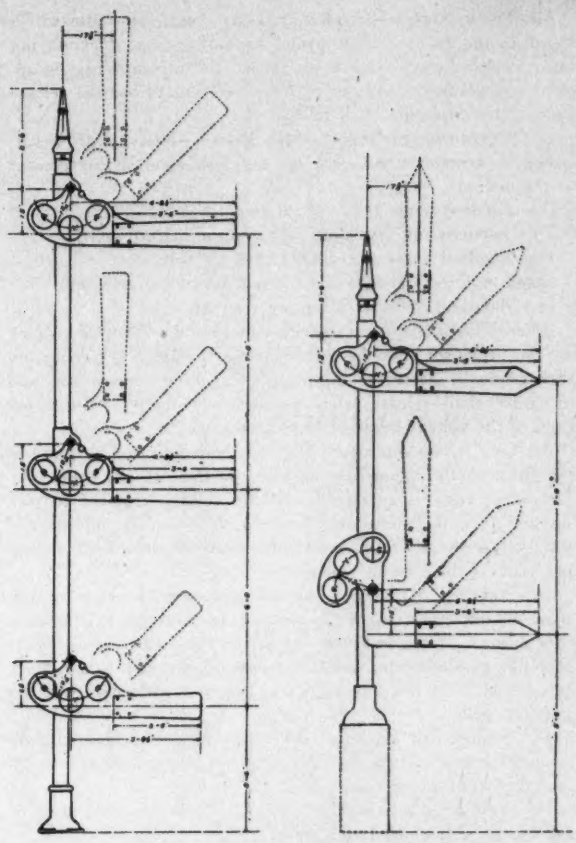


Fig. 8.—Two and Three Arm Bracket Signals Complete.

of the mast, design "A" is placed on top and design "B" below it. They can, of course, be interchanged so far as the design is concerned.

RELAYS FOR STEAM ROADS.

1. Type.—All relays must be of the enclosed type with all working parts enclosed in a transparent case.

All relays shall be sufficiently water-tight to stand a water immersion test of at least ten minutes.

2. Material and Workmanship.—Magnet cores and armatures shall be made of the best quality of soft annealed iron, and all other materials used in the construction of relays shall be of good quality, free from defects; workmanship and finish shall be satisfactory to purchaser.

3. Armature Supports.—Armature supports must be securely mounted on the magnet cores, or the armature supports and cores may be mounted on the same piece of metal, in such manner that the same relative position of armature and core faces will always be maintained.

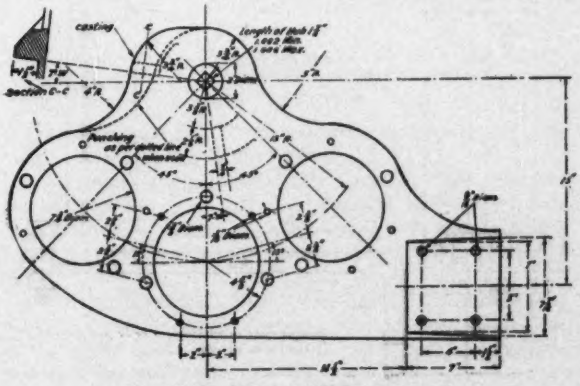


Fig. 9.—Semaphore Spectacle Design A.

Trunnions of armature bearings shall be hard drawn German silver, shall be cylindrical and at least one-sixteenth inch in diameter.

Trunnion screws must fit tightly in their supports and be provided with jamb nuts or other suitable fastenings.

4. Armature.—Springs, adjustable stops or other means of adjusting armature vertically will not be permitted.

A minimum air gap of one-sixty-fourth inch between armature and magnet cores shall be insured by bone or brass pins driven in end of cores or in armature.

Stop pins must be driven firmly to the bottom of the holes drilled for their reception and secured by cupping.

5. Contact springs or fingers shall be mounted rigidly on the armature not less than three-eighths inch from nearest face of same and shall be sufficiently heavy to retain any adjustment they are given.

6. Contact Points.—All contact points shall make sliding contact. Stationary front contact points of track relays shall be of non-fusible materials and back contact points shall be platinum. A closed contact shall not have more than 0.13 ohms resistance. All contacts shall have a minimum opening of one-sixteenth inch.

7. Magnet Coils.—All magnet coils must be convenient of application to core; all insulation shall be applied before coils are assembled on cores and when assembled they must be securely held to prevent vibration.

The wire for magnet coils shall be soft drawn copper covered with a good quality of silk or cotton braid or equally good insulation and of proper size to give resistance specified.

The ends for connecting magnets shall be soldered, noncorrosive flux being used, or may be joined with substantial accurate binding posts provided with lock nut.

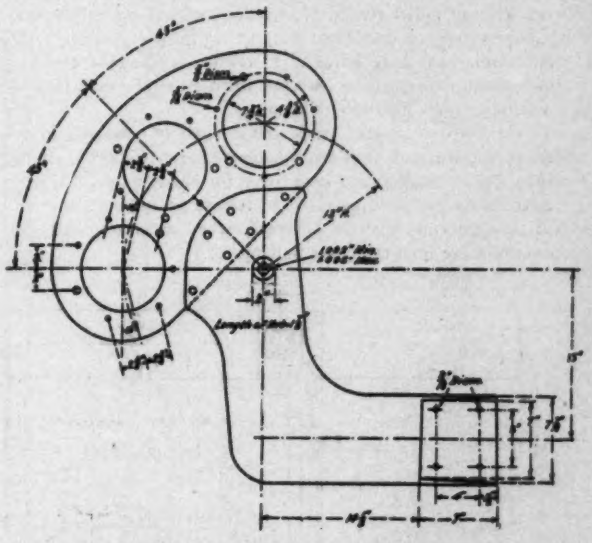


Fig. 10.—Semaphore Spectacle Design B.

Terminal wires of magnet coils shall not be less than number twenty B. & S. gage.

Magnet coils shall be encased to prevent mechanical injury to wire.

8. Binding Posts.—Binding posts shall be of brass of such construction that they will not turn in the base or frame on which they are carried. The stud carrying thumb screws for fastening wires shall not be smaller than 10-32.

9. Insulation.—An air gap of not less than three-eighths of an inch, or approved insulation equivalent thereto shall be provided between any part of the relay carrying current and any

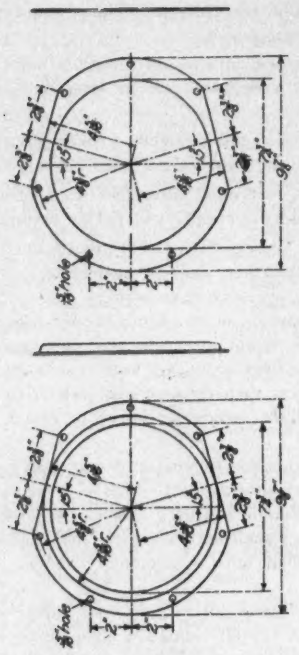


Fig. 11.—Spectacle Rings.

other part thereof. Parts so insulated shall withstand a test with an alternating current of three thousand volts for one minute.

10. Resistance.—All track relays shall be of four ohms resistance, unless otherwise specified.

11. Adjustment of Relays.—Track 4 ohms. Four ohm track relays shall be adjusted to release at not less than thirty milamperes after an initial charge of one hundred and ten milamperes has been given for one minute.

After relay has been adjusted to release as specified, the current through coils shall be reversed when armature shall pick up at not more than sixty-five milamperes.

16 Ohm.—Sixteen ohm track relays shall be adjusted to release at not less than fourteen milamperes after an initial charge of fifty-five milamperes has been given for one minute.

After relay has been adjusted to release as specified, the current through coils shall be reversed, when armature shall pick up at not more than thirty-six milamperes.

All Track Relays.—If after a relay has been adjusted as specified, the front contact points are adjusted so they do not make contact when relay is energized, the initial charge is applied and gradually reduced to forty per cent of normal release current, the armature shall release.

12. Adjustment of Relays.—Line Relays.—Relays shall be adjusted so armatures will pick up and give good sliding contact as follows:

One hundred ohms at not more than 1.7 volts.

Two hundred and fifty ohms at not more than 2.6 volts.

Five hundred ohms at not more than 3.6 volts.

Seven hundred and fifty ohms at not more than 4.4 volts.

One thousand ohms at not more than 5.0 volts.

After being adjusted as specified the voltage required to pick up the armature shall be increased one hundred per cent, and then gradually decreased by means of a variable resistance, and armature shall release when voltage is reduced to forty per cent, of the voltage required to pick up same.

All Line Relays.—If after relay has been adjusted as specified the front contact points are adjusted so they do not make contact when relay is energized, and the voltage required to pick up armature increased one hundred per cent, is applied and gradually reduced, relay shall release at not less than sixteen per cent of the pick up voltage.

13. Residual Magnetism.—Before adjusting all relays of less than one hundred ohms resistance must be subjected to a momentary e. m. f. of not less than forty volts, and all relays of more than one hundred ohms resistance must be subjected to a momentary e. m. f. of not less than one hundred volts nor over five hundred volts.

14. Sealing and Marking.—All relays must be sealed with the manufacturer's seal, and all adjustments marked plainly in the sealed portion of the relay.

15. Test after Assembling.—All tests to be repeated after relay has been assembled and sealed.

OPERATING REQUIREMENTS—SIGNALS.

a. Semaphore Casting, Mechanism and Blade Complete.—The total torque, measured at twelve inches from the center of the semaphore shaft, with four feet six inch blade, roundels and rings in place and mechanism connected shall not be less than forty-two pounds, when the blade is moving toward the proceed position and is within five degrees of it. The total torque shall not be less than twenty-eight pounds when the blade is moving toward the proceed position and is not more than five degrees from the stop position.

The total torque when the blade is moving from the proceed

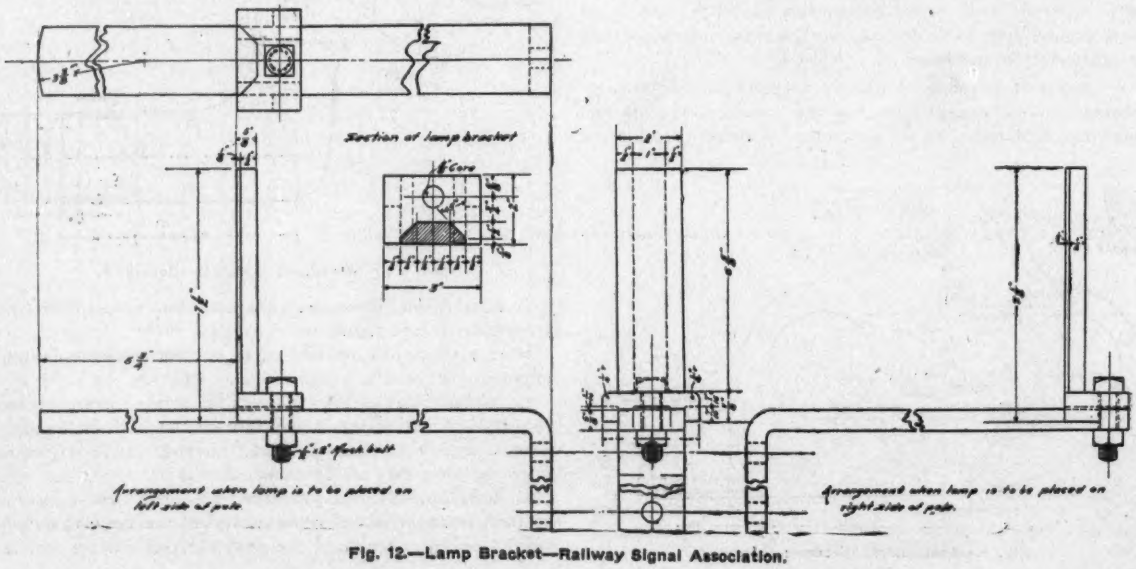

Section of lamp bracket
Arrangement when lamp is to be placed on left side of pole.
Arrangement when lamp is to be placed on right side of pole.

Fig. 12.—Lamp Bracket—Railway Signal Association.

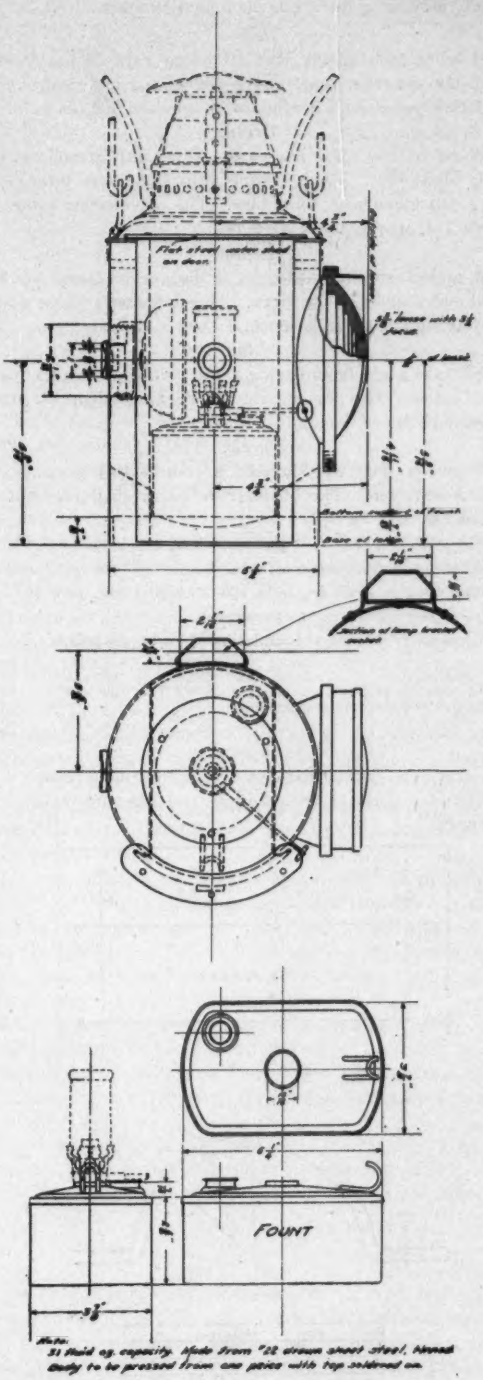


Fig. 13.—Lamps.

to the stop position shall not be less than five-sixths of that mentioned above at corresponding points.

b. Semapore Castings and Blade Complete.—The torque of the semaphore spectacle and blade complete, disconnected from operating rod or mechanism, shall not be less than two-thirds of the corresponding total torque at any point in the stroke of blade.

c. Mechanical Release Test.—When the signal is in complete working order and in the proceed position, the effective force, at the center of the "hold clear" armature, tending to release it shall not be less than two pounds.

d. Electric Release Test.—When the hold clear coils are energized and the signal is in the proceed position, the armature shall release when the E. M. F. measured across the terminals of the coils is reduced to four volts.

e. Operating Voltage, Current and Time.—The E. M. F. required to clear the signal shall be not more than six volts. The signal shall require not more than two and one-half amperes at ten volts to move from the normal to the forty-five degree position in five seconds. For any other degree of movement, time to be increased or decreased in proportion.

GRAVITY BATTERIES.

a. Jars.—Glass jars for gravity batteries shall be six inches in diameter by eight inches in height (inside dimensions).

b. Zincs.

1. Zincs shall be of the shape and size shown on railway company's plan No. .....

2. Zincs shall be made from virgin spelter, thoroughly amalgamated with mercury, cast at a low temperature, uniform in size and weight, free from flaws and mechanical defects and have a smooth outer surface. A fracture of the zinc must show the grain firm and close.

3. Weight.—The zincs shall weigh four pounds each.

4. Chemical composition of the finished zinc shall be as follows:

Mercury	Not less than	2.00 per cent
Iron	Not more than	.10 per cent
Lead	Not more than	.50 per cent
Other impurities	Not more than	.40 per cent
Zinc	Not less than	97.00 per cent

SULPHATE OF COPPER.

1. Material desired under these specifications is the normal crystallized sulphate of copper.

2. The material shall meet the following requirements:

a. It shall be entirely free from acid of any kind.

b. It shall not contain more than three-fourths of one per cent of sulphate of iron.

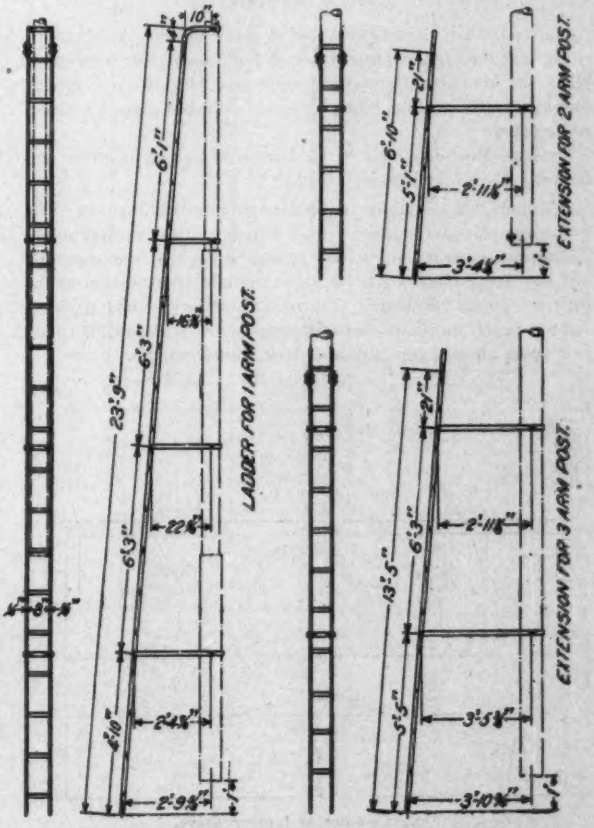


Fig. 14.—Standard Ladders for Pipe Signal Posts.

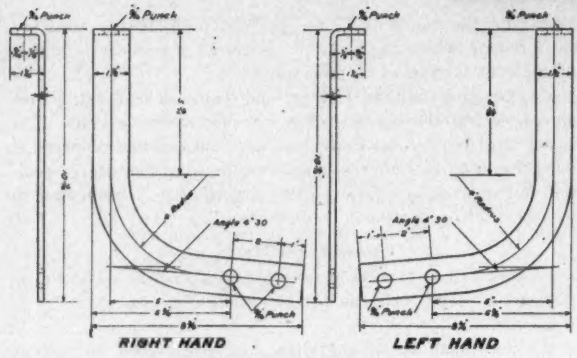


Fig. 15.—Top of Ladders.

c. It shall not contain more than one-half of one per cent of other impurities.

d. All material must pass through a sieve having one and one-fourth-inch mesh.

e. The amount of the material that will go through a sieve having one-fourth-inch mesh must not exceed five per cent of the weight of the shipment.

COPPERS.

Coppers for gravity batteries shall be in accordance with the railroad company's plan ..... All leaves shall be No. 32 B. & S. gauge, Copper must be ninety-eight per cent pure.

Lead wire shall be securely riveted to each copper with a pure copper rivet having a three-eighths-inch head and a copper washer three-eighths inch in diameter shall be placed on side of copper for riveting. Lead wire shall be No. 14 B. & S. gauge copper, must be fourteen inches long and insulated with three-sixty-fourths-inch rubber covering the entire length, except one inch at the top.

SIGNAL ROUNDELS.

Material.—All glasses must be of clear uniform solid color, containing the highest proportions of lead compatable with durability and securing the required color and must have a specific gravity of not less than 2.75. Chipped or flashed glasses will not be accepted.

Design.—Roundels must be of diameter specified in order, and between .21-inch and .29-inch thick.

Roundels will be subject to spectro-photometric analysis. The following table gives an analysis of roundels of the various colors of medium intensity, the letters indicating lines of the spectrum, and the figures showing percentages of light transmission at the different points. Roundels of medium intensity should transmit light as nearly as possible of this composition, a reasonable variation being allowed for light and dark limits.

	A.	a.	B.	C.	D.	E.	b.	F.	G.	H.
Red	60	65	70	72	0	0	0	0	0	0
Green	0	0	0	0	4	27	40	45	25	0
Yellow	0	38	50	43	41	12	9	3	0	0
Blue	0	0	0	0	3	4	6	24	40	46
Purple	0	42	42	0	0	0	0	2	43	42
Lunar White	0	62	49	17	15	25	38	65	74	0

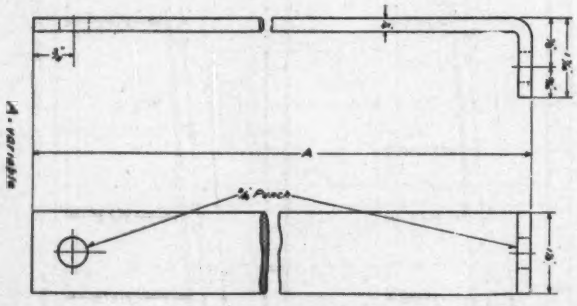


Fig. 16.—Sides of Ladder Stays.

Briefly describing the above photometric values.

Red.

Will be of such quality that all yellow rays of light are absorbed, the spectrum being either red or red and orange. The photometric value shall be, light 130, standard 100, dark 70.

Green.

Shall be of the color known as Admirality green, having a slightly bluish tint. The spectrum shall show very little yellow, being a full green with some blue. The photometric value shall be, light 125, standard 100, dark 75.

Yellow.

Will give a spectrum showing a full yellow band, most of the red and slightly of the green. The photometric value shall be value 120, light 120, standard 100, dark 80.

Blue.

Will give a spectrum having a full blue band, with a narrow band of green. The photometric value shall be, light 125, standard 100, dark 75.

Purple.

Will give a spectrum showing a considerable proportion of both red and blue. The photometric value shall be, light 125, standard 100, dark 75.

Lunar White.

Shall show a maximum of absorption for the yellow. The photometric value shall be, light 120, standard 100, dark 80.

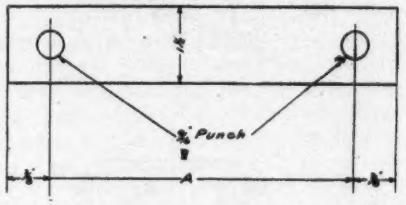
FOUNDATIONS.

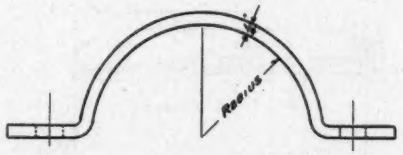
Dimensions of concrete foundation shall be as follows:

	Top.	Base.	Depth.
Dwarf Signal	30"x16"	30"x16"	36"
Straight post signal	26"x26"	36"x36"	48"
Straight post signal with mechanism in base	30"x36"	42"x48"	48"
Bracket signal	28"x38"	48"x48"	60"

CONCRETE.

(a) Concrete shall be made of one part Portland cement, three parts sand, six parts of broken stone, and water to make proper consistency.





Part	For	Radius	A
1	4" Pipe	2¼	6¼
2	5" "	2¾	7¾
3	6" "	3⅜	8⅜

Fig. 17.—Front and Back Clamp for Ladder Stays.

(b) The outer exposed face to a thickness of one inch shall consist of one part Portland cement, one and one-half parts sand, deposited at the same time as the interior mass. The top surface shall be floated and rubbed smooth by hand and shall be true to grade and line.

BONDING.

Where joints are located in platforms or road crossings, whether of plank, brick or ballast filling, four No. 6 B. & S., soft drawn, bare copper or copper-clad bond wires must be used at each joint, two being placed on either side of rail and outside of angle bar.

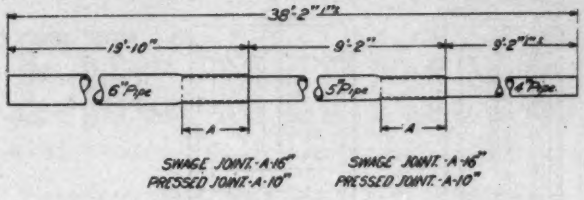


Fig. 18.—Standard Pipe Signal Post.

Frogs must be bonded to adjoining rails to make the frog a part of the circuit.

Channel pins shall be driven immediately after the rail is drilled.

GALVANIZED IRON BOND WIRES.

This wire to be used as bonds around rail joints in electric track circuits.

The wire must be of the grade known as E. B. B. and must be drawn according to Birmingham Wire Gauge (B. W. G.) No. 8.

The wire must be galvanized, cylindrical in form, free from scales, inequalities, flaws and splits, and all other imperfections and defects.

Weight per mile: Maximum, 392 pounds; required, 378 pounds; minimum, 364 pounds.

IRON LINE WIRE.

The wire must be of the grade known as "E. B. B." and drawn according to the Birmingham wire gauge (B. W G.).

The wire must be galvanized, soft, pliable, free from scale, inequalities, flaws, splits and other imperfections or defects.

Wire of No. 8 gauge and larger must not vary more than three mils from its nominal diameter, and No. 9 and smaller not more than two mils.

The wire must be straightened before winding into coils.

The wire of each coil must be of one continuous length and must be guaranteed not to contain any weld, joint or splice, either in the wire or the rod from which it was drawn.

All wire must be "killed" or stretched two per cent before coiling.

All coils must be not less than 18 inches in diameter.

All coils must be securely tied for shipment.

Galvanizing must be well done, uniform in thickness and must not crack or break off when the wire is bent around a piece of two-inch round iron.

The weight per mile of finished wire must not vary more than three per cent from the weights in the following table:

Size B. W. G.	Weight lbs. per mile	Weight required in full coil.
4	787	180 lbs.
6	573	175 lbs.
8	378	170 lbs.
9	305	150 lbs.
10	250	125 lbs.

The finished wire must not break under a strain less than three times its weight per mile and must elongate without rupture not less than fifteen per cent in ten inches. A sample of the wire when gripped by two vises six inches apart and twisted must stand without fracture not less than fifteen complete twists in one direction in the six inches.

The electrical resistance per mile must not be above that shown in the following table:

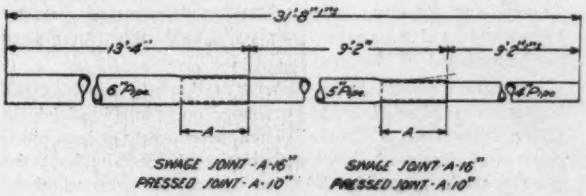


Fig. 19.—Standard Pipe Signal Post.

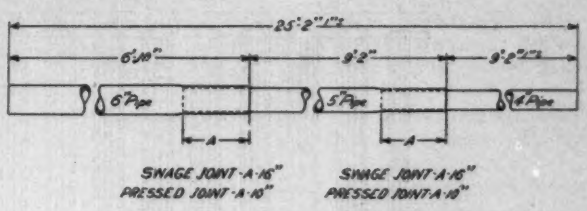


Fig. 20.—Standard Pipe Signal Post.

No.	4 E. B. B.	5.97 Ohms.
	6 E. B. B.	8.21 Ohms.
	8 E. B. B.	12.42 Ohms.
	9 E. B. B.	15.44 Ohms.
	10 E. B. B.	18.83 Ohms.

HARD DRAWN COPPER LINE WIRE.

The finished product desired under these specifications consists of hard drawn copper wire, covered with a weatherproof braid, as hereinafter specified.

The copper bars, before rolling, shall be entirely free from defects. Each coil of wire shall be drawn in one continuous length and shall be free from factory joints. The conductor shall be uniformly cylindrical in form, and free from scales, inequalities, flaws, splits and all other imperfections, and shall be drawn according to B. & S. gauge.

To insure the removal of defects from the wire, the manufacturer shall cut off at least twenty-five (25) feet of wire, or as much more as may be necessary from each end of every coil.

The conductor shall be covered with two (2) close braids of cotton. This braid covering shall be saturated with a permanent waterproof compound that shall be insoluble in water. The compound shall not melt or crack when subjected to a temperature of 120 degrees above and below 60 degrees F.

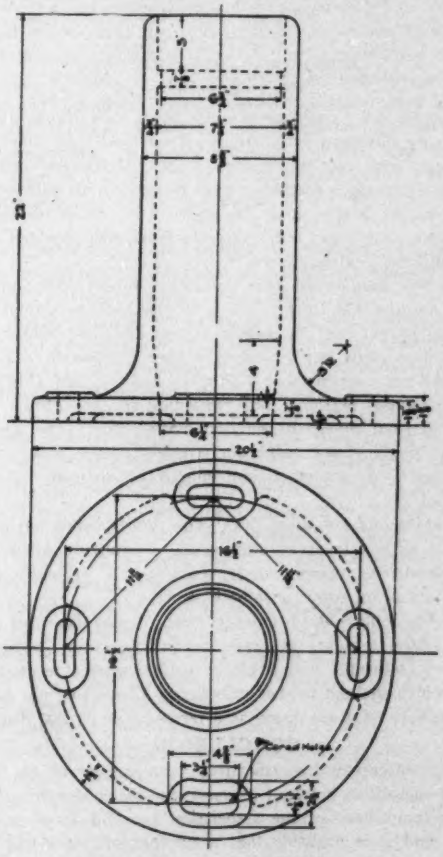


Fig. 21.—Base for 6-in. Signal Post.

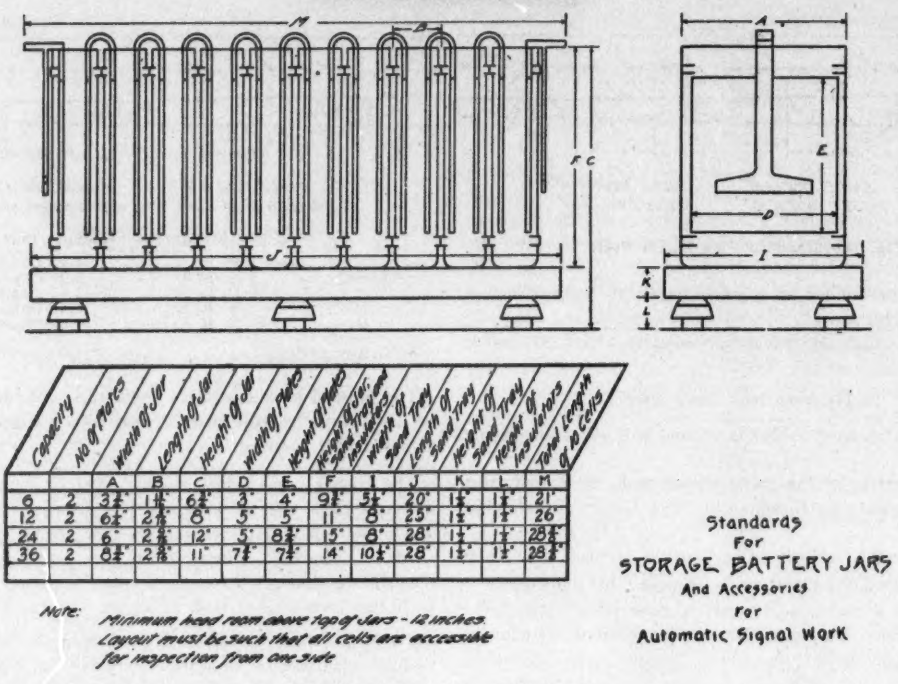

Capacity
No. of Plates
Width of Jar
Length of Jar
Height of Jar
Width of Plates
Height of Plates
Height of Jar, Tray and Insulators
Width of Sand Tray
Length of Sand Tray
Height of Sand Tray
Height of Insulators
Total Length of 10 Cells
A B C D E F I J K L M
6 2 3¼" 1¼" 6¼" 3" 4" 9¼" 5¼" 20" 1½" 1½" 21"
12 2 6¼" 2⅛" 8" 5" 5" 11" 8" 25" 1½" 1½" 26"
24 2 6" 2¼" 12" 5" 8¼" 15" 8" 28" 1½" 1½" 28¼"
36 2 8¼" 2⅛" 11" 7¼" 7¼" 14" 10¼" 28" 1½" 1½" 28¼"
Note: Minimum head room above top of Jars - 12 inches. Layout must be such that all cells are accessible for inspection from one side
Standards For STORAGE BATTERY JARS And Accessories For Automatic Signal Work

Fig. 22.—Railway Signal Standards.

MECHANICAL AND ELECTRICAL REQUIREMENTS.

Gauge	Breaking weight in pounds	Percentage Elongation	Weight per mile	Conductivity in per cent of pure copper
12	319	1.00	158	96
10	502	1.02	241	96
9	620	1.06	283	96
8	779	1.06	349	96
7	970	1.08	425	96
6	1210	1.12	529	96

RUBBER INSULATED SIGNAL WIRE.

600 volts or less.

1. *Conductors.* Conductors must be of soft-drawn, annealed copper wire having a conductivity of not less than ninety-eight (98%) per cent, of that of pure copper, Mattiessen's standard. Each wire forming a conductor must be continuous without splice throughout its length, must be uniform in cross section, free from flaws, scales and other imperfections and provided with a heavy uniform coating of tin.

2. *Rubber Insulation.* The vulcanized rubber compound shall contain not less than thirty (30%) per cent, nor more than thirty-three (33%) per cent, by weight of fine dry Para rubber which has not previously been used in rubber compound. The gum itself shall not contain more than three and one-half (3½%) per cent of resinous extract. The remaining seventy (70%) of the compound shall consist of mineral matter only. The insulation must be tough, elastic, adhering strongly to the wire, must be homogeneous in character and placed eccentrically about the conductor.

3. *Taping and Braiding.* (a) The rubber insulation must be protected with a layer of cotton tape thoroughly filled with a rubber insulating compound, lapped one-half its width and so worked on as to insure a smooth surface.

(b) The outer braid must consist of one layer of closely woven cotton braiding at least one thirty-second (1-32) of an inch thick, saturated with a black, insulating, weatherproof compound which shall be neither injuriously affected by nor have injurious effect upon the braid at a temperature of 200 degrees F.

CONCLUSION.

The above cuts and specifications do not cover all the apparatus and operations involved in automatic block signaling, but the various committees of the association are still at work on the subject and it is probable that a number of years will be required to complete the work.

Grade Crossings

While invariably avoiding grade crossings on new and revised construction work, the Pennsylvania Railroad has, in the last ten years, been eliminating all crossings at grade as rapidly as practicable. A compilation for the period since January 1, 1910, shows that 673 grade crossings have been removed on the lines of the system east of Pittsburg and Erie. These figures are of record of September 1, 1909, and do not include the ten crossings to be eliminated by the change of line to be made through Bristol, Pa., on the New York division.

On the lines of heaviest traffic between New York and Washington, and Philadelphia and Pittsburg, the company has abolished 256 public grade crossings in the past ten years. The 774 public crossings remaining are scattered over 574 miles of road and are, with a few exceptions, at unfrequented highways where traffic is inconsiderable. There have also been removed, in addition to the 256 public crossings, 129 private crossings at grade.

The completion of the new Washington Terminal represents one of the most important steps taken in the plan of the late President Cassatt to eliminate grade crossings in important cities between New York and Washington. To accomplish this has involved the elevation or depression of tracks in Jersey City, Newark, Elizabeth, New Brunswick, Trenton, Philadelphia, Chester, Wilmington, Baltimore and Washington, and work is to be started in the next few days on the elimination of the crossings in Bristol, Pa.

On the division between Philadelphia and Washington alone 106 grade crossings have been eliminated. Between Jersey City and Philadelphia 78 crossings have been abolished in the past ten years. On the Pittsburg division 46 crossings have been removed, leaving but 80 on the main passenger line between Altoona and Pittsburg. Out of 218 grade crossings between Philadelphia and Altoona, on January 1, 1900, only 73 remained on September 1 of this year.

The figures given take no account of the very extensive and important revision of grade—so conspicuous in Philadelphia—whereby tracks for trains in different directions have been either elevated or depressed, thereby eliminating the necssity for trains to cross tracks at grades. This has been a very great additional factor of safety at places where traffic is dense.

RAILWAY ENGINEERING
AND MAINTENANCE OF WAY.
BRIDGES—BUILDINGS—CONTRACTING—SIGNALING—TRACK

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Vol. VI. Chicago, January, 1910 No. 1

Beginning January

The issues of RAILWAY ENGINEERING for this year will take up the standard practice of railroad companies in automatic block signaling. The statement is made now so that our readers will see that their copies are not lost. As mentioned heretofore, the articles will be written from a practical standpoint so that they will be of value and interest to all railroad men who have to do in any way with the operation of trains.

If you have any question to ask, we will be pleasd to give same our attention. Automatic block signaling is becoming standard practice on many large railroad systems and it is thought to be only a question of time when all railroads will adopt it.

With this issue the standards of the Railway Signal Association are published. These form a basis for future articles on railroad installations and comparisons may therefore be much better defined.

Descriptions of standard signaling equipment, as furnished by manufacturers, may at a later date be published. We will wait, however, until the present series of articles are well under way before taking up this matter.

Maintenance of Way Association

The annual meeting of the American Railway Engineering and Maintenance of Way Association will be held in Chicago, beginning Tuesday, March 15th. Printed copies of the convention reports are now being sent out by the secretary, E. H. Fritch, Monadnock Block, Chicago.

The reports of standing committees are a continuation and development of previous reports. It is therefore advisable to have at least the 1909 proceedings in bound form, which will be furnished by the secretary in order to refer to same in discussing or reviewing the 1910 reports.

Our readers, who are not members of this association, would find it to their advantage to get the proceedings of the Maintenance of Way association, so as to keep in touch with standard practice. The revised edition of the "Manual of Recommended Practice" of the association may also be obtained from the secretary.

A brief summary will be given here of the committee work to be taken in March, if it is completed or in good shape for presentation.

The Committee on Roadway has for consideration the collection and tabulation of all known formulas for determination of size of waterways and the consideration of a general formula for waterway areas, unit pressures allowable on roadbed of different materials, tunnel work and drainage questions.

Relating to ballast, stone ballast physical tests are to be considered, an amplification of the specifications in regard thereto, and the best treatment of stone ballast that has become foul in the track. This committee is also to report on gravel ballast with respect to grading and to confer with Committee on Roadway in regard to unit pressures.

Under ties, the committee has for consideration the conclusions derived from compiled statistics after consideration of continued compilation. The tie supply question and that of metal, composite and concrete may be discussed again this year.

Rail specifications will be considered in view of present statistics which are being compiled from reports, mentioned in the December issue of RAILWAY ENGINEERING. Standard designs and specifications for rail joints may also be taken up.

The Track Committee is to consider with the Committee on Signaling the effect of metal track equipment coming in contact with track circuits; specifications for frogs, switches and switch-stands; recomendation of three lengths of switch points; minimum number of lengths for frogs, not above four; practical lengths for switch leads, widening of gage, and canting of rail.

Reinforced concrete for coaling stations and storage bins may be taken up, also design and arrangement of oil houses at terminals, roof coverings and designs for section tool houses.

Structural timber specifications are for consideration of Committee on Wooden Bridges and Trestles, together with pile-driving and definitions relating to specifications for timber.

For the Masonry Committee the questions for consideration are waterproofing, reinforced concrete specifications, monolithic construction in arches or large abutments wtih wing walls, plans for retaining walls and abutments, and reinforced concrete trestles.

On the subject of Signs, Fences and Crossings, the questions are the proper quality of fence wire, concrete fence posts and flangeways at road crossings.

The Signal Committee is to continue the investigation of outline and description of a comprehensive and uniform signal system, and also to consult with other committees on construction affecting signaling apparatus.

Specifications for water tanks and trestle supports, from 50,000 to 100,000 gallons capacity, are to be taken up by committee on Water Service, also friction factors for delivery pipe lines and for water columns, and designs of different types of track pans.

Typical designs of hump yards relative to traffic of empty and loaded cars particularly, mechanical handling of freight and typical layout for passenger terminal were assigned to Committee an Yards and Terminals.

The investigation of impact effect on bridges is to be continued, also flat spots on car wheels, care of existing bridges, steel versus reinforced concrete for main girders of bridges carrying heavy loads under high speeds, and review of the development of metal bridge building.

The Committee on Wood Preservation has to consider the proper grouping of timbers for antiseptic treatment, compilation of statistics in co-operation with other committees, preparation of timber for treatment, proper quantity and quality of antiseptics to be used for preservation in connection with conditions under which they should be used, and the strength of treated timber.

The Committee No. 18 on Electricity has for consideration the lateral and vertical clearances required for installations for electric traction, safeguards for line crossings over tracks, insulation, maintenance organization and electrolysis.

Numerous specials are also to give reports at the March convention. The above outline gives briefly the topics which were assigned to different committees for consideration. All of the above work may not, however, be in shape in March, so the reports on some topics will be held over for next year.

Deformed Bars vs. Round Rods Anchored for Reinforced Concrete*

Reinforced concrete is no longer a new material but has entered the engineering field to stay. With its application to all kinds of structures many systems have been invented, some of which are based upon logical principles, reliable experiments and a few years of practical results; while others are poor imitations of the pure gems and will soon drop out of sight when engineers, architects and contractors become better acquainted with their respective values.

The various systems used differ not only in their general design but more especially in the shapes and qualities of the tension members used. It is not our aim to describe these systems and the qualities of the different grades of steel used, but to confine ourselves to the advantages and disadvantages of the shapes used which are of two general classes, plain and deformed bars.

The simplest shape and the most easily gotten anywhere is no doubt the round rod. This shape has many distinct advantages, among them that of giving the greatest sectional area under the smallest volume; it is easily bent, is symmetrical about its center of gravity, has no sharp edges and can be handled rapidly without injury to the hands which feature is not to be overlooked from a contractor's standpoint.

Round rods were first used in reinforced concrete structures in Europe where they are still used almost exclusively, while the tendency seems to be towards deformed bars in the United States. The reason for this is on account of the different ways and means engineers employ to get at the same result. In the United States materials are cheap and labor is expensive so that usually, as far as possible, the cheaper grades of labor are used in building work. In Europe exactly opposite conditions prevail; materials are expensive and labor is cheap so that contractors can well afford to use a higher class of labor. Both ways harmonize perfectly with the actual conditions and both of them are to be recommended since they result in either case in a saving on the total cost of the structure.

The class of labor employed in Europe on this work, though paid only 60c to $1.00 per day as many of you probably know, is to be compared with our skilled mechanics whose wages are from $3.00 to $4.00 per day. You will now understand that it is possible to rely upon European laborers with far more certainty than can be done with the laborers employed in this country since their grade is altogether different. For this reason European engineers have usually used the plain round, rods in reinforced concrete work, which have always proven satisfactory so far as I know. Some, however, have bent the ends in such ways as to secure a better anchorage, as they were afraid of defective work. In the United States it is altogether different and we prefer to spend a little more money on materials, since they are cheaper, and also to use such a system that will leave but little to the initiative of the laborers and so insure ourselves against defective work. This, I repeat, is entirely justified on account of the widely different conditions.

It would, no doubt, be poor practice to use plain round rods without any kind of anchorage in this work unless the supervision was so perfect as to avoid any chances for doubtful work.

Inventors have come forward with numerous devices aiming to overcome such chances and the deformed bars were placed on the market by enterprising business people who saw a chance to make money, while others thought their devices to be technically perfect, often through lack of the special knowledge that reinforced concrete involves.

By deformed bars I mean bars with any kind of kinks, bumps, recesses, holes, corners, twists, notches, etc. I think that most of you gentlemen who are engaged in the building line have been misled as to the value and effectiveness of these so-called deformed bars, through error, since so many of the inventors knew so little about concrete that they are excusable for their products.

All of you know that in the design of reinforced concrete members, the concrete is relied upon to take all the compressive stresses while the steel takes the tensil stresses. These tensile stresses are transmitted to the steel bars through the medium of the concrete only and this is possible on account of the physical property known as adhesion or bond between the concrete and the steel. It is, of course, well to increase this bond to the greatest possible unit figure in order to get the best results and numerous devices have been patented which aim to a greater safety.

Some kind of anchorage, mechanically obtained, is necessary in order to secure safe results, but we should know exactly what we are doing, and what additional stresses are impossed upon the concrete by the use of these devices.

First let us see how deformed bars act when embedded in concrete. The adhesion of the concrete to the steel is mostly due to the contraction of the concrete while setting and when a tensile stress is exerted upon the bar it is resisted by two forces known as tangential and normal adhesion as shown by Mr. R. Feret* in his experimental researches recently published.

Now in a reinforced concrete beam these conditions of transmission of stress are reversed and the concrete must transmit the tensile stresses to the steel rods but the same force still are there to produce the necessary grip or bond. Besides these, other internal stresses known as secondary stresses also occur, such as the propagation of the normal and tangential adhesion through the strata of the concrete. as shown in Fig. 1, in the same way that electrical waves and magnetic forces are propagated around a pole. This also has been studied by Feret and we will not go into more detail now, but wish that all would bear these points in mind as they are most important.

The more deformed the bars, the more uncertain the secondary stresses, since the propagation has a tendency to follow the broken lines, as shown in Fig. 1, and the equation of these stresses in the case of deformed bars seems to be a mathematical impossibility, though Mr. Feret has made a study of the stresses in the case of round rods.

The mechanical bond obtained with deformed bars is due to the tendency to resist slipping through the concrete on account of the deformations. A greater resistance to slippage

*By J. H. Toupet, Ch. Eng., Electric Welding Co., from Proceedings of Engineers' Society, Western Pennsylvania.

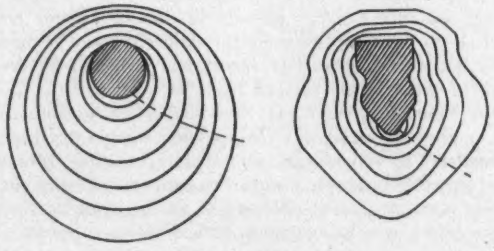

Fig. 1.—Reinforcements.

*Etude Experimentale Du Ciment Arme.

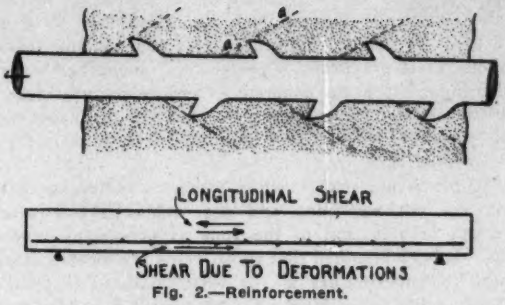

Fig. 2.—Reinforcement.

than with round rods is of course secured but at the same time, and as a natural consequence, a greater longitudinal shear obtains. This additional stress which results from the use of deformed bars constitutes the main objection to their use. This stress may vary with the shape of the bar; but it always occurs and is a minimum for the smoothest surface, that is for the round rod.

To show you a little more clearly what I mean, suppose that a bar, as shown in Fig. 2, is embedded in concrete. When a tensile strain is exerted upon the concrete to be transmitted to the steel rod the concrete will naturally have a tendency to shear through the planes *aa*, possibly not exactly those indicated, but through similar planes more or less inclined. This shearing strain is repeated at each and every deformation, varies at every point of it and also at every point of the bar in proportion to the tensile strength to be transmitted. The summation of all these shears constitutes a very indefinite strain which is a part of the additional longitudinal shear mentioned before. This stress is a direct result of the deformations. It is in fact a direct shear along the longitudinal axis of the beam. Another increase in the longitudinal shears is due to the greater bond which is secured with a deformed bar when properly embedded.

The total longitudinal shear due to pure adhesion, and to deformations, if any, must be taken by the concrete web since this stress is distributed through the concrete and the necessity of a web thick enough to resist it, is at once apparent. In other words it shows that an ample spacing between the rods should be allowed in proportion to their cross sectional area and also to the additional longitudinal shear they may develop on account of the deformations.

It means that with an increase in the bond a proportional increase in the thickness of the web should be allowed, which is never done. I might say that the contrary is true and that quite a number of people using deformed bars reduce the surrounding concrete to such a thickness that it becomes very dangerous.

If designers were willing to take into account the additional longitudinal shear consequent upon the use of deformed bars, which they ought to do, it would be very difficult to make a theoretically correct design since these stresses are unknown and a guess can only be ventured as to their value. However, it might be well to increase the thickness for the web in proportion to the bond given by the deformed bar, taking as a basis the plain round rod.

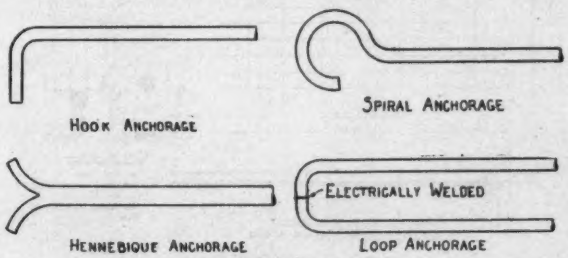

Fig. 3.—Reinforcements.

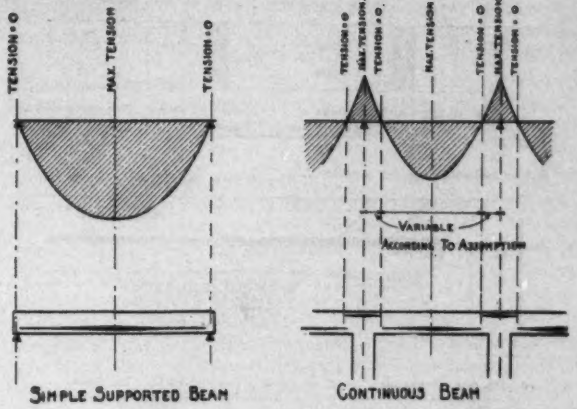

Fig. 4.—Theoretical Amount of Steel Required in Tension.

The question now arises: How much good do we get out of the use of deformed bars if we do not know the exact value of the additional stresses consequent upon their use? No engineer or contractor would, as far as I know, take a chance in other lines of construction, but they do in concrete work, for the very reason that additional stresses creep in as explained before. I understand that it would be difficult to mathematically demonstrate the extent of such stresses. All that is necessary to be thoroughly convinced of their existence is to follow a logical line of reasoning and study the experiments made by reliable authorities who are not paid for securing special results.

I would also like to call your attention to the fact that some deformed bars may produce a torsional movement in the concrete while some others are so wide and so placed that a decided breaking plane is formed through the concrete, especially in the webs of beams heavily reinforced.

To summarize the situation, deformations always produce additional stresses of indefinite value which more than offset the greater bond obtained. These additional stresses are very dangerous especially with green concrete; when the centers are struck the concrete is never perfectly set and great care should be taken that no undue strains come upon it, the stresses due to bending under the dead weight of the

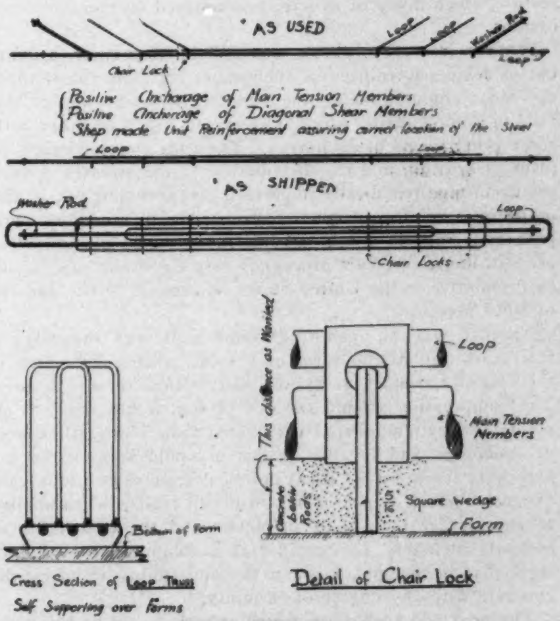

Fig. 5.—Reinforcement.

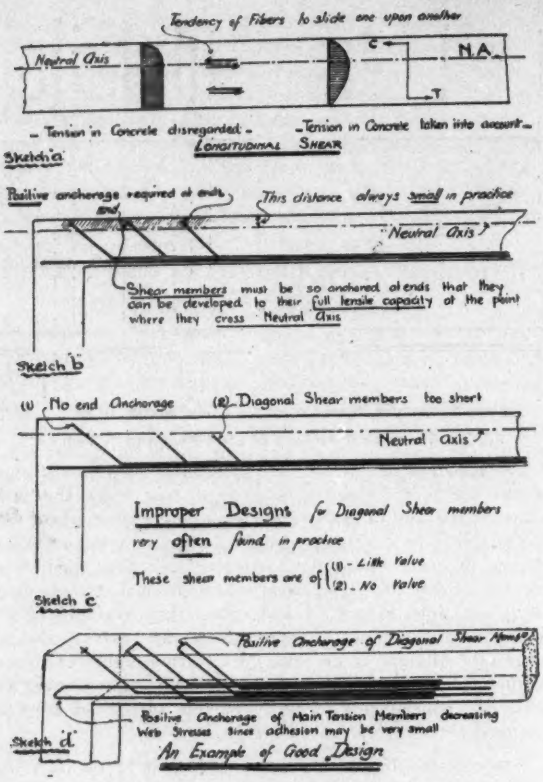
Tendency of Fibers to slide one upon another
Neutral Axis
C
N.A.
T
– Tension in Concrete disregarded –
– Tension in Concrete taken into account –
LONGITUDINAL SHEAR
Sketch a
Positive anchorage required at ends
This distance always small in practice
Neutral Axis
Shear members must be so anchored at ends that they can be developed to their full tensile capacity at the point where they cross Neutral Axis
Sketch b
(1) No end Anchorage
(2) Diagonal Shear members too short
Neutral Axis
Improper Designs for Diagonal Shear members very often found in practice
These shear members are of (1) – Little Value (2) – No Value
Sketch c
Positive Anchorage of Diagonal Shear Members
Positive Anchorage of Main Tension Members decreasing Web Stresses since adhesion may be very small
Sketch d
An Example of Good Design

Fig. 6.—Reinforcements.

structure being the only ones that should be permitted and these should be reduced to a minimum.

When diagonal shear members are made of deformed bars bent up, it is needless to say that they are of little value since they cannot be developed to a stress anywhere near that computed. These bars require a positive end anchorage to be effective as do round rods or plain bars of any shape, since their stress must be entirely developed at the point where they cross the neutral axis of the beam. Deformed bars also give a chance for air holes, a very objectionable feature which many of us have encountered on numerous occasions.

When reinforced concrete was still in its infancy, a number of devices were used as anchorages for plain round rods, the most common being the bending of both ends of the rods in a hook shape, which is shown in Fig. 3 together with some other forms of anchorage. The rods were awkward to place in position and the distribution of the stresses around the anchorage technically imperfect. Hennebique, one of the pioneers in reinforced concrete construction, used rods with both ends split in a fish tail like manner and his followers are still doing it. This process is very expensive and would be prohibitive in the United States on account of the amount of labor involved.

Another way of anchoring round rods was suggested a few years ago in the form of a spiral anchorage which is theoretically perfect as far as distribution of stress goes. The compression around the coil of rod, if not equal at all points, is very much better distributed than in any other form of anchorage and for this reason it would seem to be the best were it not for its great practical drawbacks. It is very expensive and it is nearly impossible to realize when applied to rods larger than ¾-in. diameter; and the connection of concrete members, for instance at a column, are so full of steel that it becomes a practical impossibility to place the concrete with any degree of certainty.

The next end anchorage, which appeals to me as being the best, although it is not theoretically as perfect as the spiral, is the electrically welded loop, which is made by bending both ends of the rod at right angles and welding them together. It is practically a perfect end anchorage since the rods cannot slip in the concrete under any circumstances, and the stresses consequent upon this method of anchoring are merely compressive and not longitudinal shear as in the case of deformed bars.

The bends are made with a sufficient radius so that the rods are not weakened and the welds have a practical strength of 9-10 that of the rods as proven by numerous tests. It is to be noted that these welds are located at points where the stresses are nil, or very small, so that there is no chance left for a dangerous structure. This loop anchorage, shown in Fig. 3, requires no explanation and I believe that it will appeal to all as being a thoroughly reliable anchorage. The cost is not prohibitive and the result is a very stiff unit for slab, beams, or girders.

The anchorage of tension members should always be made at their ends and not along their entire length as in the case of deformed bars, or of plain rods without bends at ends, for the following reason: The concrete is usually in greater tension near the center and at the bottom of a beam than near the supports; for instance, we all know that for a simple supported beam there are no tensile bending strains at the supports and that such strains are maximum at the center of the span, while in more or less continuous beams the tensile strains in the bottom of the beam, are nil at some points between the supports and the center of the span, which points are of variable location according to the more or less continuity of the member considered. The stresses are also reversed at the supports in the case of continuous members. The amount of steel required in simple and continuous beams is shown in Fig. 4, in a diagramatical way, which amount varies in a direct ratio to the absolute value of the bending moment. The beams are supposed to be uniformly loaded.

Of course it would be a very poor design and bad practice to stop the rods back of the supports in continuous girders, or slabs, since other stresses, such as vertical shear at the supports or tension due to a particular loading, may occur just where there would be no reinforcement to take care of it. A strong connection is always desirable at columns even at the expense of a little more steel. It is good policy, however, and effects great saving to have the amount of reinforcement vary within reasonable limits, from the supports towards the center of the span.

In the simple supported beam, shown in Fig. 4, the anchorage is made in what might be called an inert body since there are no tensile stresses in the concrete at these points and the anchorage of the rods is thus secured with a minimum stress upon the concrete. In the case of continuous members the end anchorage is made in the compression side of the beam and its value is increased since the compression

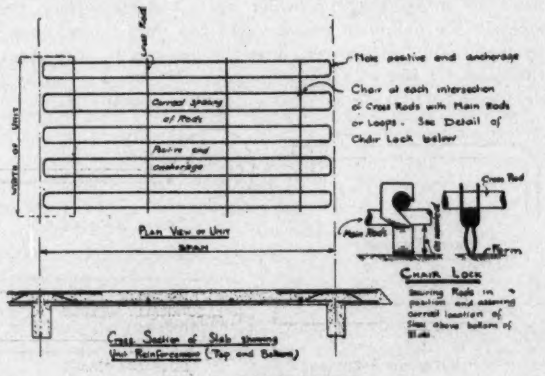
Cross Rod
Main Rods
Width of Unit
Correct spacing of Rods
Positive end anchorage
Plan View of Unit
Span
Note positive end anchorage
Chair at each intersection of Cross Rods with Main Rods or Loops. See Detail of Chair Lock below
Cross Rod
Main Rods
Chair
Chair Lock
Bearing Rods in position and assuring correct location of Slab above bottom of Slab
Cross Section of Slab showing Unit Reinforcement (Top and Bottom)

Fig. 7.—Reinforcements.

acts upon the rod as a superload tending to increase the effectiveness of the anchorage. This loop anchorage has never failed and practical tests have shown its great reliability.

Mr. R. A. Cummings has developed a system of reinforcement along the lines outlined for the loop anchorage, the principal features of which are: A positive end anchorage secured by the electrically welded loop, a shop made reinforcement avoiding misplacing of the steel, and a correct and economical distribution of the steel through the use of the loop truss shown in Fig. 5. Whenever there is too little room for the loop, a washer is welded at the ends of the rod forming a positive anchorage which absolutely prevents slipping of the rods in the concrete.

It is to be noted that the beam is of varying cross sectional area and that is a very economical design. The diagonal shear members are part of the main tension members which is very desirable and they are also securely anchored at the ends by means of the loop. These diagonal shear members are, as previously pointed out, of little use if not securely anchored at the ends. In this loop truss the diagonal members can be developed to their ultimate strength without slipping, and I do not know of any other design as safe as this one. The loop truss is a shop made unit, it is shipped flat, is easily placed in the forms and is self supporting over the forms.

In sketch *a*, Fig. 6, is shown diagramatically the value of the longitudinal shear in a beam when the tension in the concrete is either neglected or taken into account.

This distance from the top fiber to the neutral axis being always very small, as shown in sketch *b*, Fig. 6, usually about 2-10 of the total depth of the beam, and as the diagonal tension members reach in practice to but within 2 in. from the top of the beam, it is evident that a positive end anchorage of the ends of the diagonal shear members is absolutely required since these must be developed to their full strength at the neutral axis.

Sketch *c*, Fig. 6, shows some improper designs very often found in practice. Diagonal tension members which are a part of the main tension members, are a required feature, but they must be anchored at the upper end and are of little value at all if too short.

Sketch *d*, Fig. 6, shows an example of a correct design: The loop truss with the diagonal tension members an intregal part of the main tension members, and thoroughly anchored at the upper ends by means of a loop or an electrically welded washer. There is no possible slipping of the rods through the concrete.

It is needless to say that round rods can be combined in a thousand or more different ways to thoroughly correspond with the engineer's calculations. It is our practice to design a beam for the particular loading it has to carry as the shear members are variable in most every case. The loop anchorage, as shown in Fig. 7, is used in making units for slab reinforcement and has proven very satisfactory in every case.

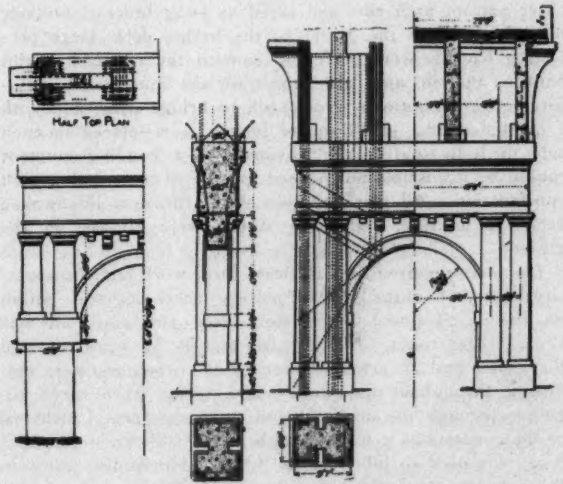

Fig. 1.—Main Bridge Piers, Mulberry Viaduct.

The question may be asked why so many deformed bars are sold and why quite a few are a commercial success. This can be answered in a very few words: The American people take the lead for their business ability, their ceaseless energy and their admirable methods of advertising, which latter often makes a success of an undesirable product. Contractors are often willing to consider the cheapest as the best no matter what the final results may be, and this is the real explanation for the success of a few products which, if they prove to be a good financial result, are a very poor engineering success.

The Mulberry Street Viaduct, Harrisburg, Pa.*

The Mulberry Street Viaduct is a reinforced concrete highway structure of the rib arch type. It consists of a main bridge 1,841 feet long and 60 feet maximum height, spanning Paxton Creek Valley from the center of Harrisburg to the suburb of Allison Hill, and crosses Tenth Street, Cameron Street, Paxton Creek, and twenty-seven tracks of the Pennsylvania Railroad and Philadelphia and Reading Railway. At the western end, 600 feet is on a 12-degree curve and the remainder on a tangent. An inclined approach, 600 feet in length, are carried on arched piers about 40 feet in height at about the middle of its length. The bridge replaces a steel structure built in 1891 and considered unsafe for the increased traffic of to-day.

The bridge was designed and the construction supervised by Mr. James H. Fuertes, Consulting Engineer, of New York city, and is believed to be the most advanced development of the rib arch in this country and the first attempt to apply this type to a continuous structure of such magnitude.

The main bridge has a roadway 28 feet wide and two sidewalks each 8 feet wide, and the Cameron Street approach has a roadway 21 feet wide and one sidewalk 6 feet wide. Supporting the deck are ten segmental arches of 100 feet span and fourteen arches of spans varying from 40 feet to 96 feet. The main bridge has grades of 1 and 3 per cent and the Cameron Street approach grade is almost 7 per cent. The design was complicated by the lack of headroom over the railroad tracks and by the necessity for placing all piers on the curved portion of the bridge parallel with the tracks and oblique to the superstructure, causing a number of skew spans. The spans of the main bridge, 100 feet in length, are acrried on arched piers about 40 feet in height from the ground to the skewbacks.

Near the center of the bridge is a massive abutment pier designed to take the unbalanced thrust of the Cameron Street approach spans and serve as a landing for this approach. There are three abutments with retaining walls and earth fills, the east abutment, on Allison Hill, about 75 feet long, the west abutment, 150 feet long, and the Cameron Street abutment, 100 feet in length.

The main bridge spans consist of four reinforced concrete arch ribs, without openings, and serve also as spandrel walls for the deck support. The deck consists of a continuous slab of reinforced concrete supported on transverse floor beams which are carried by the spandrel walls. The slab cantilevers beyond the outside ribs and is strengthened

*From a paper by J. E. Allen and B. G. Love in Proceedings of Engineers' Club of Philadelphia.

by ornamental brackets. The roadway is paved with asphalt and the sidewalks are granolithic. The railings are of wrought-iron pipe supported by concrete posts. The posts over each pier are enlarged to form a base for the lamp-posts, which are also of concrete. Drainage was provided by grate inlets over each pier with outlets of cast-iron pipe carried down through the piers and discharging at their base. The Cameron street approach is similar in design to the main bridge except that the deck is carried by three arch ribs instead of four.

A continuous line of reinforcing bars was placed in each rib along the entire length of the bridge immediately below the level of the deck, and this reinforcement was heavy enough over each pier to support the two adjacent half spans as cantilevers. After the bridge was completed from abutment to abutment, these bars were cut over each pier, which allowed each span to become an arch. The specifications required that arch construction should be carried on without subjecting the piers to an unbalanced thrust.

The bridge was designed for a uniform live load of 200 lbs. to the square foot over the entire roadway, 150 lbs. per square foot over the sidewalk area, and 40-ton electric cars on two tracks. The floor slabs are proportioned for a local concentration of 1,000 lbs. per square foot and the floor beams provide for maximum shears of 24,000 lbs. The concrete is proportioned for maximum stresses of 500 lbs. per square inch where reinforced and 300 lbs. per square inch where not reinforced. The reinforcement was stressed to 16,000 lbs. per square inch. All foundations were carried to hard rock.

Bids for the construction were received July 5, 1907, and the contract was awarded to McCormick & Co., of Philadelphia, on July 30, 1907, at unit prices for the various materials and operations, reaching an approximate total of $262,000. The quantities included 17,000 cubic yards of excavation, 21,000 cubic yards of concrete, 600 tons of steel bars, 2,000 linear feet of cast-iron pipe, 5,000 linear feet of pipe railing, 5,000 linear feet of electric duct, 7,000 square yards of asphalt paving, and 44,000 square feet of surface dressing. Allen & Love, of Philadelphia, were retained by the contractors as consulting engineers.

Field work was started by the demolition of the existing bridge. The Etter Erecting Co., of Philadelphia, was awarded the subcontract for this work and began operations on August 6, 1907. The old bridge consisted of 100-foot spans of pin-connected deck trusses and 70-foot spans of through Pratt trusses with cast-iron compression members. The flooring was of timber. The work was accomplished by a two-boom traveler on the bridge deck, the structure being supported by light timber bents at the panel points. The material removed amounted to approximately 350 tons of structural steel and 240,000 feet B. M. of lumber. This work was somewhat delayed on account of the difficulty of obtaining a right of way over the railroads and making satisfactory arrangements for the protection of railroad traffic, but the operation was successfully concluded by October 15, 1907.

Office and shop buildings were placed under construction August 3, 1907, and the working plant assembled. The buildings consisted of a main office, engineer's office, blacksmith shop, a fully equipped sawmill, a cement storehouse of 2,000 barrels capacity, a stable, and three buildings for the storage of small tools and supplies. A large platform for laying out full size form details and a well stocked lumber yard completed the layout. The working plant comprised six derricks and hoisting engines, five concrete mixers, four 6-inch steam pumps, four portable boilers, an air-compressing plant, a 10-ton road roller, and the large variety of small tools necessary for such a construction. The maximum force employed on the work was 200 men and an average force of 100 men was carried throughout.

Construction of the new bridge was started on the foundation excavation for the abutment of the Cameron Street approach, September 1, 1907. Excavation was through fill, clay, an water-bearing gravel to rock at depths varying from 20 to 30 feet. Water was found at a depth of 10 feet in all excavations and in such quantities that a 6-inch pump could not maintain a condition fit for placing concrete. Outside sumps were tried and the sheathing calked without much effect, and it was found necessary to enlarge the excavations and build water-tight forms, which were practically cofferdams, inside the sheathing. With this, it required pumping night and day to control the water until concrete was placed above the water-line.

This work on the eastern section of the bridge was simple in comparison with that on the western section. Here, in addition to the water, was found a sliding soil, which required heavy shoring to prevent the railroad tracks from caving into the excavation. The volume of railroad traffic was another obstacle, and all tracks were required to be kept open, although the clearance between the cars and the work was not sufficient for standing room for a workman. The excavated material was raised by a derrick mounted on a flat car and loaded directly on cars for removal.

The foundation concrete was 1:3:6 mixture, the sand used being dredged from the Susquehanna River near Harrisburg. The use of this sand was discontinued above the neat line on account of the quantity of coal dirt which it contained, samples yielding as much as 30 per cent at times. For the remainder of the work, sand was procured from the west branch of the Susquehanna River about 40 miles above Harrisburg, and proved very satisfactory.

The pier foundations, from 6 inches below grade to rock, consist of two shafts 8x8½ feet in cross-section and 31 feet apart on centers. They are enlarged at grade to form pedestal bases 8½x11 feet and 4 feet high. Above these bases the cross-section is 9½x6½ feet, ornamented with taper pilasters and a cornice. The shafts are connected above grade by a semi-circular arch 21-foot span forming an ornamental portal. Supported on these shafts is a girder 12 feet deep, 7 feet wide, and 45 feet long, on which the arch ribs are seated. This girder contains about 133 cubic yards of 1:2½:4½ concrete. It was required that it should be molded monolithic, which required about sixteen hours of continuous work. The reinforcement consists of 31 twisted bars 1 inch square along the bottom. The triangular skewbacks 5 feet 6 inches on the base, 2 feet 9 inches in height, and 4 feet 6 inches long, were molded on top of and with the girder. A mat of steel bars was placed in their upper surface. Above the girder and extending to the under side of the bridge deck, curtain walls 2 feet thick connected the outer pair of arch ribs and acted as sway braces. Outside of the ribs from the girder to the bridge deck, large projecting backets were placed to conceal the expansion joint between the ribs and form a base for the lamp-posts. Reinforcing bars, continous from rock to bridge deck, and with a combined area of 15 square inches, were placed in each side of both shafts and provided for a bending moment caused by a possible unbalanced load of 20 tons during arch construction. All pier concrete above the neat line was a 1:2½:4½ mixture, and large stones were allowed in the girders.

The piers required very intricate form work and the necessity for monolithic construction increased the strain on them. Sectional forms were used and each set was shifted three times. About 15,000 feet B. M. were required for a pier and an average force of 35 carpenters were employed throughout the work. The forms were wired together through the concrete and the braces were anchored to the concrete as it was placed by bolts with the nut end in. Soap was used to lubricate the forms. Horizontal joints in the concrete were made by placing large stones projecting

above the surface, and before work was resumed the surface was washed and flushed with cement grout. The concrete was mixed wet and thoroughly tamped and spaded. Bottom dump buckets were used in placing it.

The finished piers are very rich and imposing in appearance, while retaining the lightness in keeping with the entire design. They contain an average of 400 cubic yards of concrete each, and it took about three weeks time to construct one. The entire surface was hammer-dressed. The work was handld by guy derricks with 90-foot masts and 85-foot booms, so located that they controlled the work on two piers each.

On the western section of the bridge the portal effect was not used on account of the low piers, but the general construction was similar throughout. Here the ground was not firm enough to support the girder forms, and they were built within timber trusses, with a clear span of 23 feet, supported directly on the pier foundations. The flooring was suspended below the truss by ¾-inch bolts, and the construction being monolithic made it necessary to camber these trusses to overcome the deflection. The piers contained about 324 cubic yards of concrete.

Owing to the lack of clearance through the railroad yards, it was necessary to have the pier forms self-supporting. No inclined braces or shores could be used and the forms were built in a crib work of heavy timbers securely bolted together. There being on ground space for a concrete plant, recourse was had to a traveling plant. The mixer was placed between the tracks at the end of the pier and the derrick was mounted on a flat car, coupled in a train with supply cars of cement, sand, and stone and moved in on a track adjacent to the work. Whenever the track occupied was needed for travel, the entire train was backed onto a siding and returned when the way was clear. Fairly rapid progress was made considering the annoying but unavoidable delays and conditions, concrete being placed at an average rate of 80 cubic yards per day. Piers through the railroad yards contained about 350 cubic yards of 1:2½:4½ concrete and were 20 feet high, 8 feet wide, and 51 feet long.

The east abutment is built on the rock face of Allison Hill. The face wall is 30 feet in height and not reinforced. Pockets were formed in the face wall to receive the arch ribs.

In the rib arch construction the abutment walls are built simply as retaining walls, the ribs being continued as buttresses along the line of pressure to solid foundation. These buttresses were built monolithic with the walls, terminating in skewbacks within the face wall, and contained reinforcing continuous with the arch rib reinforcement. The West and Cameron Street abutment design was similar. Tongue and groove vertical expansion joints were placed about every 30 feet in the side walls.

The abutment pier, about the middle of the bridge, is 41-82 feet square and 64 feet extreme height, containing about 3,500 cubic yards of concrete. It is of cellular construction and the only reinforcement is the continuation of the rib reinforcing through the buttress walls. The exterior and the buttress walls are 5 feet thick at the bottom and are offset to 2 feet thick at the top, inclosing twelve rectangular wells about 16 feet long by 6 and 8 feet wide. The exterior is paneled and molded to conform with the remainder of the superstructure.

When excavation was started for this pier, the impossibility of draining the entire area was apparent, and the excavations for the exterior walls were carried down first, a side at a time, and concrete placed to above the groundwater line, which process, when completed, formed a concrete cofferdam, effectually excluding the water and allowing the interior work to be carried on without difficulty. This foundation concrete was placed during freezing weather, but a straw and tarpaulin covering proved a sufficient protection.

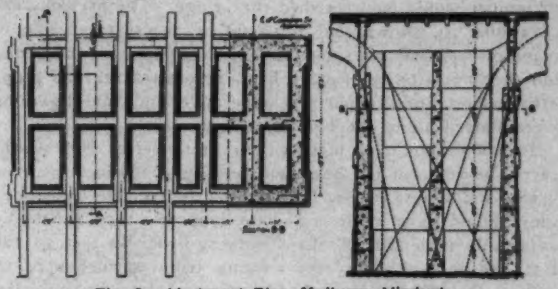

Fig. 2.—Abutment Pier, Mulberry Viaduct.

Above grade the interior walls were built first with projecting keys to unite with the exterior walls. The forms were wired together and also braced with inclined shores to give stability for the great height. The exterior walls were granolithic faced and the entire pier hammer-dressed.

The piers of the eastern section were ready to receive the arches by July, 1908. These consist of four reinforced concrete ribs 3 feet deep at the crown, 17 feet deep at the skewbacks, and 12 feet apart on centers. The two outer ribs are 24 inches thick and the two inner ribs 26 inches thick. The intrados is a curve of a radius equal to the span and a rise, on the 100-foot spans, of 14 feet. The ribs are combined with the spandrel walls, having a horizontal upper surface at the sub-grade of the roadway. Each rib is reinforced with 1-inch twisted bars continuous from out to out of supporting piers, one-half just below the extrados at the crown and diverging from the lower bars, passing over the skewbacks 11 feet apart. In addition, there was the line of bars in the top of each rib, giving the cantilever action, which required 14 square inches of steel over the pier in each rib for a 100-foot span. As the weight of a 100-foot span exceeded 1,000,000 lbs., no special provision was made for wind stresses, impact, or vibration, but to avoid long, unbraced compression members, the lower edges of the arch ribs are connected at the quarter points by horizontal slabs 10 feet wide and 6 inches thick.

A variation of 5 inches in the length of the bridge was calculated as the result of temperature changes, and allowance for it was made by expansion joints over the center of each pier. A corrugated break was made between the abutting ribs for a height of 7 feet over the skewbacks, and there widened to 2 feet and extended to the bridge deck. This pocket allowed access to the cantilever reinforcing bars and was sealed at the roadway sub-grade by concrete slabs.

The deck consists of a continuous slab of reinforced concrete, 8 inches thick in the central portion of the span and 6 inches thick at the ends, reinforced longitudinally with ½-inch twisted bars 9 inches apart in both the upper and lower surfaces. The slab is supported by transverse floor beams 19 inches deep and 9 inches wide, spaced 4 feet 6 inches on centers and reinforced with three ¾-inch twisted bars. The roadway slab cantilevers 4 feet beyond the outside ribs and is strengthened by ornamental brackets. The ribs, deck, beams, and brackets were built monolithic in day's-work sections the full width of the bridge, finished against radial bulkheads. All arch concrete was 1:2:4 mixture with the sand item composed of one part sand and one part stone dust. A 100-foot span contained 333 cubic yards.

The specifications and design of the bridge required that the arches should be erected as balanced cantilevers. The original intention was to use the usual timber centering and preliminary studies were made. The method of construction required that three complete spans of centering should be provided in order that work might proceed without interruption. This would have required 400,000 feet B. M. in the trestles and 42,000 feet B. M. for lagging and deck forms. A conservative estimate of this in place the first time was $22,000, and it is doubtful whether three times this quantity

of lumber would have finished the bridge. The previous excavations had shown that it would be necessary to pile in many places to obtain a satisfactory foundation for the bents, but the impossibility of using bents across the railroad property made a new departure imperative and investigations were started along the line of steel trusses.

The conditions to be met by the centers were: they should carry, in addition to their own weight, a 100-foot span of bridge weighing 1,000,000 lbs., without appreciable deflection; they should have no intermediate supports; they should be adjustable in length to spans varying from 50 feet to 100 feet and in width for ribs varying from 22 inches to 26 inches; they should be arched to provide clearance over the railroads, but must not deliver any thrust to the piers; they must fit skew as well as regular arches; they should be light enough in their component parts to permit moving from place to place; and should be cheap enough to compete with timber construction.

All truss designs proved too expensive, and plate girders were tried. These too were excessive in both cost and deflection until advantage was taken of the method of arch construction and they were made into cantilevers balanced over the piers. This brought the cost very near to that of timber work, but the only way to make them actally economical was to dispense with some of the form work, and this was accomplished by splitting the girders longitudinally through the center of the web plate, moving the two halves apart the thickness of a rib, laying a floor within them on the line of the arch, and molding the concrete within the girders and against the web plates. This reduced the form work to the deck lagging only, and this was carried on the top flanges of the girders. The variation in the length of the spans was provided for by shortening the cantilevers and suspending a central girder between them, from which sections were cut to suit the span under construction.

Then came the question of providing supports for the end reactions of 565,000 lbs. It was necessary that these should be supported directly from bed rock, and there were the bridge piers already provided and proof against settlement. There was no footing on the piers suitable for the purpose, but the designer had left several openings through the curtain walls. Through these openings were passed beam girders overhanging each side of the pier, and a beam girder along the face of the pier was suspended by bolts from these cantilevered beams, forming an adjustable footing for the centers.

The girder forms were proportioned for the heaviest rib of 26 inch thickness and 100 feet span. This rib, with its area of deck, weighed 300,000 lbs. The steel necessary to support this was estimated at 34,000 lbs. and the wood deck forms were approximated at 22,000 lbs., giving a total weight to be supported of 356,000 lbs. Subsequent calculations raised this to 380,000 lbs., making a reaction at each bearing point of 190,000 lbs. As construction was to be monolithic and by day's work finished against radial bulkheads, an assumption was made that 50 cubic yards could be placed per day, and the arch was divided into 14 spaces containing approximately 25 cubic yards each. As the load must be balanced over the pier, this gave 50 cubic yards as the day's work. These fourteen spaces gave a series of load points of 30,000 lbs. each, on which calculations were based.

There were three conditions of loading during construction. First, on leaving an abutment it was necessary to apply the complete load when one side only was a cantilever and the remainder a supported beam. This gave the maximum stress at the center of the span of 243,000 lbs. As the deflection here would have been excessive and the girder was not braced against buckling, a light bent was provided with a bearing ⅛ inch below the girder in order to allow that deflection under load and obviate any danger of splitting.

Case two, which occurred in every span, where one side was a cantilever and the remainder a supported beam and the cantilevered side loaded to the center of the span. This gave maximum stresses at the reaction point, the cantilever flanges and ties, and the connection to the middle girder. The reaction at the loaded end was 189,000 lbs., the stresses in the cantiliver flanges and ties was 303,000 lbs., and the connection to the middle girder was 193,000 lbs.

The third case, which also occurred in every span, was a complete load with both ends acting as cantilevers. This gave stresses below those of cases one and two, and was useful only for calculating deflections. Case two gave the maximum deflection of 9-16 inch at the outer end of the untied cantilever, but as this point was raised when the cantilever was tied, the deflection became a mean between this and case three, or about ⅜ inch. This was shown to be correct by actual performance, the deflections varying from ¼ inch to ⅜ inch, and in only one case was this exceeded by a maximum of ½ inch.

As the concrete arch itself was to be constructed as a self-supporting cantilever, there was no doubt that the centers would be relieved of a large amount of the load by their own deflection. What this relief would amount to could not be positively figured, but no hesitation was felt in placing a unit stress of 32,000 lbs. per square inch on the steel for purposes of calculation.

The cantilevers were made 22 feet 6 inches long, 4 feet deep at the outer end, and 12 feet deep at the point of support. The top and bottom flange angles were 6x6x9-16 inches and stiffeners were placed about every 4 feet. The web plates were 5-16 inch thick and ⅝ inch rivets, countersunk in the webs, were used throughout. The support was a rocker bearing made from two 6x3½x¾ inch angles.

The girders were 4 feet deep and 45 feet long with 6x6x7-16 inch flange angles and 5-16-inch web plates. As the varying lengths of spans required these girders to be cut thirteen times, the stiffeners were spaced to suit these cuts and the flange angles were drilled at the shop for all these connections. The only field work necessary to shorten the girders was to cut the flange angles with a hack saw and chisel down the web along the face of a stiffener.

The bottom flanges of both cantilevers and girders were connected by stay plates at intervals of about three feet. These plates were drilled in multiple to suit the varying widths of ribs.

The adjustable flooring for the arch consisted of ⅞-inch tongued and grooved lagging on three lines of 3x8 inch yellow pine joist, cut to radius and supported as follows: 5x3x⅜ inch angles were bolted on each side through the stiffeners, in a vertical position. The 5-inch leg was outstanding and was drilled for two series of 1-inch bolts. Connecting these angles were two 5x3x5-16 inch angles with the 3-inch leg forming a shelf to receive the scarf boards. Adjustment to the different widths of ribs was secured by using alternate series of notes in the outstanding angles.

The top flanges were kept in alignment by 1-inch bolts and pipe separators spaced about every 4 feet and molded into the ribs. To allow for deflection of the centers without putting a load on the portion of the arch already in place, these pipe separators were made 2 inches inside diameter, allowing the bolts and centers a play of 1 inch free of the concrete. No further provision was made against buckling of the top flanges. The pipe separators were made with ring ends and the main body of the pipe was allowed to remain embedded in the concrete.

Connection between the cantilevers and girders was made by 5½x1x4 feet 6 inch splice plates on the flanges, secured by twenty-four bolts 1 inch diameter. To avoid jamming, thin wedges were inserted between the abutting ends of the flanges, which, upon removal, allowed the girder to swing free.

To comply with the method of construction, required that three complete spans of centering should be provided, and in order that work should not be delayed it was necessary that one span should be shifted while concrete was being placed in the other two. This meant that the cantilever must be removed from one side of the pier without allowing the tie to its mate on the other side to be slackened. On the top flanges of the cantilevers were riveted sections of 50-lb. T rail milled down to form a latch. Across these latches and through the concrete rib was placed a latch bar of two 8-inch channels with the flanges turned in. These channels were separated ⅛ inch by wood blocking to allow for collapsing them prior to removal from the concrete. The outer ends of these bars were connected across the pier by two 3½-inch dlameter bolts. Thus, when a cantilever support was lowered, the latch slid past the bar and released the cantilever tie, leaving the bar bedded in the concrete and supporting the pull of 300,000 lbs. from the cantilever still in place. The bottom thrust was taken directly through the concrete rib by a stepped bearing of wood blocking backed up by angles and plates connected to the bottom flanges of the cantilevers. The protruding steps left on the ribs were hammer-dressed away after the centers were lowered.

The overhang of the bridge deck was carried by light brackets of 3x3x¼ inch angles, projecting 5 feet beyond the rib and supporting a 6x4x⅜ inch longitudinal angle which held the outer ends of the bracket forms. These supporting brackets were bolted to the stiffeners of the outside ribs and were spacd about every 8 feet.

The cantilevers weighed complete 5.47 tons each and the 45-foot girders 5.51 tons, making a total per rib of 16.47 tons and a total per span of 65.88 tons. This was increased 2.5 tons by the sway rods and miscellaneous pieces, making 68 tons per span and a total for three spans of 204 tons.

The outer ribs being placed at the edge of the pier allowed no chance of support at that point, and the nearest opening through the curtain wall being several feet inside of the rib, made it necessary that the support for the outside rib be a cantilever with an overhang of 3 feet. The next rib, 12 feet distant and 9 feet inside the point of support for the cantilever, was 3 feet away from another point of support. The arrangement was similar for the other half of the pier, making a combination of a continuous beam and a cantilever, 40 feet in length. The load of the outer rib was 170,000 lbs. and the inner rib 190,000 lbs., giving a maximum reaction at the support nearest the cantilever of 255,000 lbs. This required the supporting member, stressed to 20,000 lbs. per square inch, to be a beam girder of two 24-inch 80-lb. I beams.

These girders were suspended by four bolts 3 inches diameter, with double nuts top and bottom, bearing on 18x18x1½ inch steel plates. The heaviest load on a single bolt was 255,000 lbs., giving a tensile stress of 16,000 lbs. per square inch.

The bolts were carried by four beam girders of two 24-inch 100-lb. I beams crossing the top of the pier through the openings in the curtain walls and resting on plates 18x21x1 inches to distribute the load to the concrete. These girders were also cantilevered, and the load being applied equally both sides of a pier kept them in balance. The necessity for removing the centers from one span before the adjoining one was completed upset this balance and required that the overload should be taken by a tie to the pier concrete. This was supplied by 3½ inch diameter bolts with nuts engaging 12x18x1½ inch plates embedded four feet in the pier cap. After the centers were removed, the bolts were screwed out. leaving the nut and plate embedded.

During part of the construction it was necessary that the cantilever foot should be carried at varying heights above the

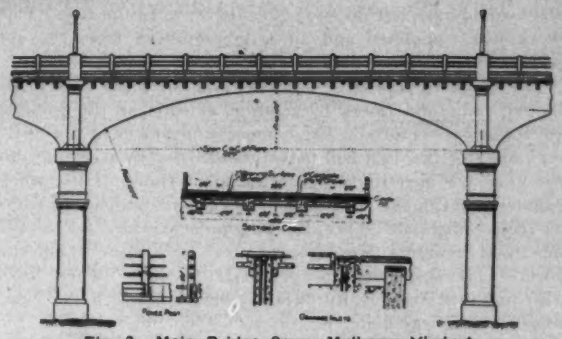

Fig. 3.—Main Bridge Span, Mulberry Viaduct.

supporting girder, and wood blocking, consisting of three pieces of 12x18 inch yellow pine capped with an 18x2x38 inch steel plate, was utilized. This was secured to the girder by two 1-inch bolts passing up between the blocks and with their heads countersunk in the plate.

The ribs were not the same distance apart throughout the bridge and on the curved portion they were not even parallel. Steel sway bracing could not be used to fit all conditions and timber sways were made into panels and spaced about 15 feet apart. Turnbuckle rods were used at each sway panel to tie the ribs together and hold them to the panels. Crossed turnbuckle rods connected the cantilevers together on the bottom flanges to prevent side deflection and kicks.

Ninety-four tons of steel were required for the supports, making a total of 300 tons required for the entire construction.

The contract for the steel was placed with the Pennsylvania Steel Co. on May 1, 1908, and it was delivered on the ground by June 30, 1908, at a total cost of $16,000.

An attempt was made to obtain a comparison of cost between the steel and timber centering, and a 40-foot span adjacent to the central abutment pier was constructed of timber. The result was not of much value as a comparison, but was very useful in convincing the adherents of timber construction that it was almost impracticable to build this bridge in such a manner.

The Cameron Street approach, with its start at grade and gradual rise to the bridge level, offered an ideal place to try out the new construction, and erection was started here on July 6, 1908. The approach having but three ribs to the span, allowed the entire four spans to be erected at once. To save time, the rib sections were raised separately and assembled in the air, but were not disassembled again until the completion of the last arch. The approach piers had been built before the centers were designed, and the anchor plates and nuts were not embeded in the pier caps, necessitating the girders crossing the piers being held down by bolts passing down the face of the pier cap and engaging other beam girders crossing beneath the cap, forming a steel strap inclosing the entire cap beam.

The deck forms were constructed in the sawmill while the steel was being erected and were placed as soon as a span was completed. They consisted of a series of boxes, placed 9 inches apart, to form troughs for the floor beams, and were supported on 2x12 inch joist which rested on the top flanges of steel forms. The sides were sloped to give a draw all around and the edge strip around the top was loosely tacked to aid in removal.

The spaces over the piers not covered by the steel were closed in with wood between the cantilever ends the full height of the ribs, the reinforcement placed, and arch concrete started August 1, 1908. Concrete was all placed and the first centers removed by September 15th, and erection was under way on the main bridge. The curtain walls were molded immediately following the completion of the arch and previous to lowering the forms, as the removal of the

steel would have left no sway bracing between the ribs. This work was completed and all forms removed from the approach by September 25th.

The forms for the large brackets over the piers were made in one piece and raised to position as soon as the wooden rib form was removed. The compression braces at the quarter points of the arch had been provided for by molding into the ribs 6x6 inch timber carrying steel stubs. This, when removed, left in the rib a 6-inch socket with projecting stubs to receive the slab. Through the pipe separators, left in the rib by the centers, were passed steel bars from which the forms were suspended for molding the slabs. These bars also provided support for the staging from which the hammer-dressing was done.

On the main bridge the supporting beams were placed first, anchored to the pier, and the footing blocks adjusted to level. The cantilevers were then raised complete by two gin poles, one at each end, and placed on their seats. The outer ends were held in position by the gin poles until the middle girder was raised and bolted fast. The wedges were then placed in the joint, the shear bolts inserted, and the end stiffener angles drawn together. The gin poles were removed to the next span, the sway frames placed and the ribs drawn into alignment with the turnbuckle rods. The rib flooring was placed to a template drawing in each section previous to erecting, and the steel was ready for the deck forms as soon as the connections were made. The tie bolts connecting the latch bars were drawn tight while jacks placed between the abutting cantilevers at the point of support raised the center of the span, thus insuring true cantilevers without arch action. The time consumed by this method for erection was four days for a 100-foot span.

The deck forms were placed from the piers outward and concrete was started two days behind the carpenters. The form work occupied eight days and the concrete keystone was usually in place on the eleventh day. The reinforcement was very heavy, and the men who had to spade concrete at the bottom of a rib 2 feet wide and 17 feet deep had no easy task to work their way through the bars. It also complicated the erection of the bulkheads for each day's work. These consisted of two 6x6 inch timbers in each rib, set to a radial line and holding a series of stop boards which were placed as the concrete was laid. Blocks on the top boards formed 2-inch depressions in the face of the concrete to serve as shear keys for the next section.

Concrete was mixed on the ground and raised on the deck by the 90-foot derricks, where it was deposited in portable bins of about 2 cubic yards capacity. From these, it was drawn off into two wheeled carts holding 5 cubic feet, wheeled where required, and dumped into place. It was now seen that the estimate of 50 cubic yards per day was too low, as double this quantity could have been placed, but it was not thought advisable to increase the size of the voussoir sections beyond the eleven spaces. As the success of the construction depended on the bond of the reinforcement for the cantilver effects, tests were made of each day's work by bedding ⅜-inch bars in the concrete to a depth of 6 inches and pulling them at intervals. When they indicated a resistance in pounds per square inch of surface sufficient for the main reinforcement to support the weight of another section, the work was proceeded with.

The wood deck forms were stripped two days behind the placing of concrete sections and the centers were lowered three days after the keystone was placed. Steel cables were attached to each of the four corners of a girder and led upward, through openings left in the bridge deck, to geared winches resting on the concrete deck. Then the shear bolts and wedges were removed, the tie plates disconnected, and the girders lowered to the ground.

The cables were then attached to the cantilevers, the supporting girder lowered by slacking off the nuts on the 5-inch bolts until the latches disengaged from the bars, the cantilevers skidded outward past the girder and lowered to the ground. Four days were occupied by this work on the 100-foot spans. On the eastern section of the bridge, with this method of working, 700 linear feet of deck was built in ninety-nine days, including Sundays and rainy days, an average rate of progress of 100 feet in fourteen days, the work being started on September 20th and completed December 27th. Arch concrete was proceeded with during freezing weather by covering the deck with canvas and laying steam-pipes inside the steel centers. The overhang was protected by canvas curtains and coke ovens were suspended from the sidewalk brackets. No difficulty was experienced in maintaining a temperature in excess of 40 degrees F.

When the deck was completed from the east abutment to the abutment pier, the reinforcing bars giving the cantilever action were cut over the piers, allowing the spans to become arches. A man with a hack saw was stationed at each arch rib over the first pier and one reinforcing bar was cut simultaneously in each rib. The men then moved to the next pier, cutting one bar, and continued thus to the last pier. This operation was repeated until all the bars were cut. There was no perceptible elongation as the acting cross-section was reduced, but the last bars snapped when cut partly through, although they did not separate much. The deflection at the center of the span was very slight; in fact, was within the possible error of measurement, but in one case a drop of ⅛ inch was observed. Possibly no harm would have been done to the structure if these bars had been left uncut, as the amount of steel in the pits free of the concrete could have elongated under the temperature stresses, but it was thought unwise to leave the structure a combination of indeterminate stresses due to both cantilever and arch action.

On the western section of the bridge the railroads became a helper instead of a trouble-maker. The ribs were assembled complete on flat cars, hauled to the desired location, and raised bodily into place by the wrecking derrick of the railroad. A complete span was erected in this way in two hours' derrick work, traffic was not interfered with, and all tracks were kept open as required. The last arch was completed on June 5, 1909, eleven months after the first arch concrete was placed.

Each of the 300 tons of steel in the centers was shifted seven times, a total of 2,100 tons of steel erected and removed again, and the concrete supported by them was 8,000 cubic yards. The total amount of lumber used in the entire construction was 800,000 feet B. M., giving some idea of the quantity which would have been required if the steel centers had not been used.

The bridge railing consisted of concrete posts 10 inches square and 5 feet 6 inches high carrying three lines of wrought pipe. Holes for ¾-inch bolts were cored in the posts on the line of the pipe and ornamental sockets placed on each side and secured by a bolt connection through the posts. The posts were spaced 9 feet on centers and anchored in place by the fascia curb being built around their bases. The fascia was 20 inches high, with a paneled exterior surface, and was secured by quills of ⅜-inch steel, which were placed every 18 inches when the bridge deck was molded. Expansion joints were provided at each hand-rail post and alternate sections were molded in succession. The roadway curbs were 18 inches high and 6 inches thick, located 8 feet inside the line of railing and simply rested on the deck. The curb and fascia inclosed a cinder fill 14 inches deep, containing electric conduits, and carrying a 4-inch granolithic sidewalk. The roadway had a cushion of gravel 6 inches thick carrying the asphalt paving on a 4-inch concrete base. At each pier, and mounted on the large pier brackets, were concrete posts 6 feet high and 3 x 2 feet square, which formed a base for the lamp-posts.

These were also of concrete. Cluster lights were placed

at each abutment entrance and at the junction of the Cameron Street approach with the main bridge, the intermediate lights being single.

At Tenth Street and at the east abutment, concrete stairways were built from grade to the bridge deck, supported on concrete brackets molded onto the piers.

The asphalt paving was started on June 15, and completed July 11, 1909, and the completed bridge was thrown open to travel August 6, 1909—just two years from the time the old bridge was closed.

Electrification of Chicago Railways

C. A. Seley, Mech. Engr.

The present controversy in Chicago over the electrification of the steam railroads has a number of features that are particularly exasperating to the partisans, both for and against the measure.

One argument advanced by those in favor of electrification is that all smoke and gases will be done away with and Chicago would become a smokeless city and that health, comfort and cleanliness would be greatly enhanced thereby.

It is assumed that as there is or is to be electrification of railways in New York, Baltimore, Detroit and other cities, as well as abroad, there is ample precedent and no great engineering difficulties in the way. The fact that Chicago railways have successfully elevated their tracks in spite of dismal prophecies regarding that undertaking is cited as an example of what the railroads can do in the way of gigantic undertakings.

The cheerful optimism of the electrical advocate, who will see no difficulty, mainly because of complete and absolute ignorance of the primary and fundamental requirements, let alone the higher and more involved elements of the problem, has been fed by a newspaper campaign of misinformation, one-sided argument, political issue, and exaggerated ridicule, which must be peculiarly exasperating to the railroad managers, who with clearer vision see more of the ramifications of the problem.

It is, no doubt, exasperating to the electrical enthusiast that the railroad man cannot see it his way; to understand that the Chicago problem is not comparable to those other electrifications that have been accomplished with apparently so little of difficulty. Because it will cost some few millions is no argument at all; that is a mere bagatelle. The railroads are used to spending millions, and the expenditure would be followed by economies and increased earnings that would not only justify the expenditure and at the same time make a smokeless Chicago with all the advantages appertaining thereto.

The smoke then seems to be the main issue, and it is charged that Chicago smoke is mainly made by the railroad locomotives. At a Coal Conference held at the University of Illinois last spring Mr. Paul Bird, the City Smoke Inspector of Chicago, made the statement that 85 per cent of Chicago smoke was made by domestic use and the small boiler plants, and he pleaded particularly for investigations and tests and the dissemination of information as to how to properly set and fire the small boiler plants of from 5 to 100 horse power, so that their stacks would not emit smoke. It is generally understood that in large boiler plants, automatic stokers, larger grate surfaces, closer attention and inspection result in a reduced proportion of smoke as compared with the small plants where automatic stokers are not generally used and hand firing is not always of a high order of excellence.

Mr. Bird said not a word at this conference regarding locomotive smoke, and, if we add an appreciable percentage as representing the smoke of large boiler plants to the 85 per cent already quoted, it leaves a relatively small amount chargeable to the locomotives. The statement of Mr. Bird is further proven by observation.

The general atmosphere of Chicago on early Monday mornings in warm weather when domestic heating is not required is very different from that later on, as the manufacturing plants get into play or work rather, and their stacks get active. Now, during Sunday, while the plants are not active, there is little cessation in railroad activity. True there are not quite so many Sunday trains, but not enough are taken off to account for the difference in the general atmosphere. Another example, take the beautiful suburban towns around us. They have as much or more railroad service as we have proportionate to their area, but their beautiful clear atmosphere is due to absence of manufacturing and heating plants. No one would claim that the smoke of South Chicago is due to the railroads, and, if the steel plants there are not smoking, there is no work for the men or benefit to the community.

As a matter of fact, these smokeless cities are not cities of industrial development. Civilization has manifold uses for fire, or, to put it another way, fire has made civilization in a large measure. The savage has little use for fire, and the lower the grade the less it is used. The higher we go in the scale of civilization, culture, and intelligence the more use is made of the gifts of nature, and our day has witnessed a great development in the use of fire for heat, light, and power, a pound of coal now producing much greater effort by reason of improvements in the machinery in which it is utilized than in the days of our forefathers.

The railroads have not been in the rear of this march of progression. Faster speeds, larger cars, and heavier trains and the thousand and one associated factors have been developed to fulfill the requirements in moving the public and its property, as the demands of travel and trade require. On all railroads, however, the best and the poorest, those in densely populated districts as well as those traversing the almost deserted plains use the steam locomotive with coal as fuel, except certain localities favored with oil, which lends itself as a convenient locomotive fuel when obtainable.

As a matter of fact, the steam locomotive has no equal as a motive power machine for general railway service. It is a moving power-house, carrying its own provision of fuel and water for considerable distances between replenishing stations. The prosperity of the country largely depends on economic movement of its people and products, and our country leads the world in these respects. It is unnecessary to go into detail to prove this statement nor the further one that the present development of the steam locomotive has been a large factor in making this possible. We were, however, considering smoke questions, and will later take up other features of the locomotive.

There are many power plants in the city, which are burning many times as much coal per hour as any locomotive, but they discharge their smoke and consumed gases high up in the air from tall stacks. The deterioration of the air, however, is just the same for the amount of coal burned, so that the influence on the general atmosphere of Chicago is proportional to the coal burned whether in a stationary plant or in a locomotive, as the discharged gases are practically the same in chemical composition.

It is true that the locomotive discharges its smoke and gases into the atmosphere at a height of 16 feet or thereabouts above the rail, instead of from 50 to 200 feet or more, as is common in stationary plants. It is also true that at times the combustion may not be perfect on a locomotive, resulting in so-called black smoke, but the railroads will admit that this can be controlled to a certain extent.

It has been impossible in the large locomotives now demanded to proportionately decrease the smoke as has been done in large stationary plants, although the efficiency and economy per ton mile for fuel has been considerably in-

*Presented before Western Railway Club.

creased, and the improvement also applies in the matter of smoke when in regular duty.

It is particularly difficult to avoid makinge smoke on our crowded terminals where signals call for unexpected stops, and then heavy acceleration is necessary to keep moving, so as not to block the tracks.

It has been stated that there are 650 smoking locomotives defiling the atmosphere of Chicago. This may be true as to number, but most of them are in and out and working much less than an hour within the city limits. This, of course, does not apply to switch engines, many of which work day and night in switching yards and transfer service. It is the smoke from these engines which is the hardest to control and requires the most watching, but after all is but a minute fraction of the smoke of Chicago.

No doubt the people who select their residence close to railway tracks find the smoke a nuisance, but, unfortunate as these may be, the smoke question must be considered as a general one, the influence on the atmosphere of the city at large.

These people have selected their homes close to the railroads in most cases for convenience of transportation, and it can hardly be claimed that they do not get it.

Those farther away do not get the convenient transportation, neither do they get the smoke, except as contained in the general atmosphere over the city, and, if our proposition has been correctly stated, the railroads are responsible for only a very small proportion.

It is believed that the effect of the smoke and gases discharged by high stacks has been lost sight of in considering the general smoke situation. When it is considered that almost every business block has its boiler plant for heating and various power purposes, and that the aggregate of these for every city block in the business district runs up into the thousands of boiler horse power, is it any wonder that the atmosphere is not clear in the business district?

Then consider the heavy manufacturing plants and power stations just a little farther away with their thousands and tens of thousands and in one case nearly one hundred thousand of boiler horse power, all consuming soft coal, is it any wonder that the atmosphere of the city is not clear?

Then the thousands of apartment buildings and residences with soft coal furnaces and ranges, the shops and small factories each contributing its quota of smoke and gases, added to the above, is it not impossible to have a smokeless city, regardless of the railroads and their locomotives?

It is presumed that the power for electrification of railroads would be generated at some outside power house, where its smoke would not be objectionable to the city. Why should not the stationary steam boilers which are used for power purposes in the city also be banished along with the locomotives and outside power generation be required? This would seem consistent if the police power of the city extends over all smoke production. It would seem as though the minor offender is the one punished in this case.

It would take too much time to go into the argument on the position of the railroads, considering their prior right of occupancy of their right of way; the advantages accruing to the city and its citizens, its business and trade by reason of the extensive railroad intercommunication which we enjoy. Suppose for one moment the situation if the steam railroads were stopped at the city limits and all transfer of passengers and freight was to be handled by street car methods, what effect would that have on the prosperity and growth of Chicago?

The panacea for all this trouble is said to be electrification, but how little is really known by its ardent advocates of the engineering difficulties to be met if the proposition of banishing steam locomotives from Chicago is to be seriously considered. The advocates say it has been done elsewhere, why not here, and thereby prove their lack of information. Every railway electrification of any size to date has been primarily for the benefit of the people in the cars, and these cars operate in tunnels. Many travelers in and out of New York blessed the day when steam locomotives were replaced by electric, and the long, disagreeable, choking Park Avenue tunnel was relieved of the smoke.

The tunnels in Baltimore are not entirely clear, but are vastly improved as to comfort in travel through them with electric power. The ability to take trains through tunnels by electric power instead of having to employ car floats is used at Sarnia and soon will be at Detroit. Some big tunnels in the Rockies are now, and others will be operated electrically as will also the Pennsylvania R. R. tunnel entrance into their New York station. The New York electrifications quoted are for handling passenger service only. When not in tunnels the roads are in a protected right of way with no freight trains, switching yards, nor industrial tracks to complicate the movement. The handling and distribution of freight at New York is practically the same as before the electrification. Both freight and passenger trains are pulled through the other tunnels cited, but the electric engines used do not go far from the tunnel portals.

As a matter of fact, they cannot. It is a common thing to see pictures of the New York Central monster electric locomotives marked 2,200 horse power, and they can exert that power for the few miles they run. It would be impossible, however, for them to take a heavy train from New York to Albany on the regular schedule of the steam locomotive. The explanation is simple. The armature of any motor must be provided with such ventilation or chance for radiation that its temperature will not rise beyond a certain degree. The armatures on these electric locomotives are such heavy bodies of metal with so little chance for ventilation or cooling that they cannot run far before reaching the temperature limit. There is no such limit to the steam locomotives and it is common to run them upward of 200 miles before changing.

The Chicago limits would not impose long distance running of electric locomotives, but the above information is given to correct the popular impression of the general feasibility of electric traction in the present state of the art.

It looks easy to propel a street car. The trolley pole is put up and a turn of the controller starts the car and controls the speed, but every car has its motors and controller, and when we consider it is common to put an aggregate motor capacity of from 50 to 150 horse power on ordinary street cars and that the New York subway cars have 300 horse power of motors and a small proportion of trailers is used, one may get a faint glimmer of what electrical traction may mean in the movement of the thousands of trains and tens of thousands of cars that are moved in Chicago daily when the motive power must be concentrated in large measure in locomotives.

It is, of course, feasible to equip Chicago suburban trains for electric traction, but the proposition is the entire elimination of steam locomotives, and the suburban movement is a small proportion of the car mileage in Chicago. If the problem was to be divided into factors of relative importance, they would be about as follows:

1. Distribution of loaded and empty freight cars to the hundreds of depots, team tracks, industries and transfers.
2. Handling of freight cars in switching and distributing yards.
3. Main line transfer freight trains.
4. Division passenger train movement.
5. Suburban passenger train movement.

The above order may be varied somewhat on different roads, due to local conditions, but fairly represents the conditions to be met on most of the Chicago railways.

Most of our roads make common use of their tracks for passenger, freight, and switching movement, and in most cases it will be impossible to work them otherwise. All of these tracks would have to be equipped with electrical transmission, which in the present state of the art is only by the third rail or the overhead conductor.

The third rail is almost impracticable to consider on account of the hundreds of open crossings, switching yards, industrial and team tracks that the public now have access to and cannot be deprived of without an almost entire rebuilding of the Chicago railway systems.

The overhead trolley wire, while overcoming some of these objections, introduces others of almost equal gravity. The complication at crossings and leads in switching yards where many tracks are often involved, the heavy transmission cables and apparatus required, most of which is located on overhead bridges, spanning the tracks, makes a chain every link of which must work perfectly or else something happens. We may be accused of looking for trouble that may not happen, but why not confess that this matter of electrification is yet all too new for anyone to guarantee immunity from trouble when considering so large a problem as this one?

Trouble has a manifold meaning on a railroad. To the man down the line it may be caused by a poor fit, a hot journal, something worn out, papers lost, lack of information, or any one of a thousand things that never gets to the manager. His troubles are things of greater amount, and over and above profit and loss is the nearer question of life or death, safe travel for patrons and employes who go in or on the cars, which must have a motive power to propel them to destination. The manager has been brought up with the steam locomotive, has seen its development, knows its limitations, provides strength in tracks, bridges, and structures with a view to all these things, so that profitable traffic can be safely transported.

Presto! Change! In an incredible short space of time measured by that usually allowed for developments of such magnitude, all the 650 steam locomotives in Chicago are to be replaced by electrics, and the 1800 or more miles of track are somehow to be equipped with electrical transmission, so that the current is everywhere available and a simple twist of the wrist will start a car or a train, and there will be no smoke!

Suppose for a moment that the roads would have no difficulty in procuring and installing all this electrical apparatus, who is to assume the responsibility for the safe conduct of the business of the railroads under the new conditions of motive power. There is as much of a problem here in getting the proper classes of talent and force to design, install, and operate this new system of railroading as it would be for the city to require the abolition of automobiles and the use of flying machines because Wright, Latham, Curtiss, Bheriot, Zeppelin, and others have successfully navigated the air and because automobiles are dangerous to get in front of and they sometimes smoke.

The prominent railway officer who is credited with saying that the railroads would electrify as soon as it would pay them to do so did not mean this in a sordid, financial sense only, but with an appreciation of the necessity for a financial return after abandoning the usual and well known methods of transportation for a new and untried system for which at the present time there is neither men, money, nor material, precedent or probability within the scope and time allowance of the proposed Chicago ordinance.

The newspapers make much of the question of economy of electrification to the railroads, basing this on statements made in technical society discussions. These statements bear no evidence of including in the comparison of costs of operating steam and electric locomotives the fixed charges incurred in the electrification, which in the New York cases must be extremely heavy, but of which there is no authentic public information. Whatever these may be applies to a passenger and suburban movement only, no freight included, so that for Chicago new and important factors must be considered.

In the statements referred to savings are claimed for locomotive repairs and the fixed charges connected therewith. This would no doubt be true to a certain extent, as the moving mechanical parts of an electric locomotive are simpler than of a steam machine. They are all too new yet to have accurate data as to the amount of saving.

Savings are claimed for less dead time for repairs and inspection. This is conceded for the reasons stated in the preceding paragraph, except that it has been shown that electric locomotives cannot be continuously operated, unless designed and operated with a view thereto in which case the quoted costs and savings would not apply. Greater ton mileage is claimed, but this is a question of comparative design and operating conditions. The data is based on short hauls in passenger service, there being no long distance heavy freight electric haul in existence.

Saving in locomotive ton mileage is claimed, but again this is a matter of design. Electric engines do not require a tender as does a steam engine, a factor in their favor. There are radical differences of design and weight in the present electric lomotives at New York City, and the Pennsylvania R. R. design will be unlike these two.

There is an undoubted saving in operation as to the cost of the power at the drawbar for the reason that one large plant of any kind can produce cheaper than a number of small plants, and a modern high grade power house can develop power much cheaper than a steam locomotive can, despite the fact that a modern, high grade locomotive is really a machine of very high efficiency. To utilize this cheap power, however, it is necessary to install a complete and complex system of transmission with its mains, feeders, contact members, converters, transformers, safety appliances and apparatus now foreign to steam operation.

It is admitted that electric locomotives cost about double that of steam, and to get at the final results of profit or loss in making this proposed change it will be necessary to capitalize the following items:

Cost of locomotives.

Cost of power house, ground and equipment.

Cost of transmission lines and apparatus between the bus bars at the power house and the contact shoes of the locomotives, including supporting structure, housing, etc.

Cost of returning current to power house.

Cost of revision of signals, telegraph and telephone lines.

Cost of revision of tracks, bridges, etc., and safeguarding the public.

Cost of new suburban cars, motors, and apparatus, if individual or multiple car service is to be established instead of electric locomotive traction.

Add sinking fund for depreciation and interest for funding the above investment at not less than 10 per cent, which may be insufficient.

Add for current expense:

Cost of the organization to design, install, and operate electrical equipment and power house, including extra supervision not required for steam operation, but not including enginemen, except as extra crews may be required for suburban motor car service.

Cost of fuel and supplies for power house, locomotives and transmission.

Deduct value of steam locomotives displaced.

Deduct value of coaling and watering stations released.

Deduct value of proportion of repair shops and facilities and their supervision released.

Deduct cost of fuel, water, and supplies for steam locomotives.

Above items to be divided between the capital and current expense accounts, as they may belong.

There are a number of contingent items not included to go to one side or the other of the account dependent on the situation such as yard and station lighting, heating of passenger equipment cars, etc. The item of cost of repairs is omitted, as no data is available.

When there is a saving in operation for the total annual ton mileage equal or better than the annual charge against the electrification, based on its cost as above computed, then the railroads can afford to banish their steam locomotives as a profitable investment.

The money question in connection with electrification is a very important one. Whether profitable or otherwise, there is no doubt but that electrification of all railroad terminals in Chicago will cost some hundreds of millions of dollars. The railroads have not got this money, but will have to raise it by the same financial methods used when making extensions, buying equipment, etc., and these funds finally come from the people to whom railroad securities are attractive as an investment. It is extremely doubtful if capital would be interested in an enterprise of any kind not able to demonstrate its ability to make financial returns, and until this matter has been thoroughly canvassed by competent engineering talent as to its feasibility and results, it cannot be financed. A bad investment reacts on the poor man's invested savings as well as on the rich man's surplus, and both are interested in knowing what may reasonably be expected. It is impossible to recall any instance of so tremendous an outlay as will be required by this proposed ordinance with so little data to give assurance that it would not almost amount to confiscation of the property.

This brings up another point for consideration. The present interchange of business and equipment between the various roads frequently requires a common use of the rails of the different companies for switching and transfers, so that an electric scheme should be practically the same for all the roads in order that these movements will not be interfered with.

This might not affect the details that any of the roads might deem necessary for their particular line, but the fundamentals and general scheme must be on such similar lines as to secure the proper amount of inter-communication. There was a time when freight and passengers had to be transferred at each railroad terminal, but in our day this is done away with by agreement between the railroads, so that a carload of freight can go unbroken from Maine to California or from Canada to Mexico, and many of these cars are handled on Chicago terminals. It will, therefore, be necessary that the electric scheme be arranged so as not to interfere with interchange.

It is suggested that the wise thing to do before any ordinance is prepared that a commission be appointed, part of its membership to consist of such a number of railroad representatives as the roads may elect, but each road to have one vote for each mile of track involved. The city to have such representation as may be desired and empowered to cast votes equal in number to the total railroad vote, each thus having an equal number of votes, and two-thirds of the total number of votes should be necessary to carry any motion involving expenditure. A board of experts covering electrical, mechanical, and transportation questions should be appointed, not less than three for the railroads and three for the city, to work under the direction of the commission, but to have no vote in matters involving expenditure. The city and the railroads should each furnish their members on the commission and their experts without cost to the other until a working plan is agreed upon.

It may be alleged that such an organization would produce no results; that the railroad would simply vote against any proposition and block progress. Possibly so, but how else can the railroads be shown the feasibility of electrification? The burden of proof is on the city, and if their experts after having full knowledge of the real conditions which can only be accurately reported to them by the railroad experts, can then present a feasible scheme to the commission the record of the action of that body may then be taken to court if necessary to decide the merits of the case.

This paper has not been prepared with a pessimistic disbelief in the possibility of electrification. One would be a fool to make any such statement. If it is granted for the moment that electrification is more expensive than steam railroading just as electric light is more expensive than gas, oil, or candle light, the fact may be shown that railroads are voluntarily electrically operating some tunnels, not that they are saving money thereby, but that they can afford the extra cost in view of the comfort of travel and safety gained in operation and making some operations possible that would not be otherwise. In this enlightened and progressive age we are doing lots of things that a while ago were not deemed possible or advisable. We are living better, or at least spending more money than our forefathers did. We travel more and indulge ourselves with many things which a while ago we did not think we could afford. They were then luxuries, now they seem to be necessities. So with electricity on a railroad. First we used it in a de luxe sort of way for lighting, and as it got to be more generally understood, better and cheaper systems were developed, and now it has become so common and reliable that some roads use it instead of gas on many of their cars.

Some day we may see our way clear to a complete terminal electrification. We don't see it now and will have to be shown. But it should come rationally and intelligently with due time for investigation and development as has been wise and necessary for great things that have been accomplished in other lines.

Let us be rational, not hysterical. Rome was not made in a day, neither can Chicago solons work miracles. This generation has seen such a rapid evolution in transportation facilities that perhaps the railroads should accept as a compliment to their ability the expectation of the people as to their ready accomplishment of electrification. It seems a short time, but how long did it actually take for the change from the old horse and cable cars to the modern high speed electric street and suburban systems, and yet can any of them compare with the speed, reliability and comfort of steam railroading?

Fairbanks-Morse Gasoline Passenger Car

The International Smelting & Refining Company have just purchased from Fairbanks, Morse & Co., one of their No. 24 motor cars for passenger service on their newly constructed line between Tooele and Tooele City, where their smelting plant is located, on the Tooele Valley Railway.

This car is of the type recently sold by Fairbanks, Morse & Co., to the Pennsylvania Railroad Co., for passenger service on the Smyrna Branch of the P. B. & W. Ry., between Clayton and Smyrna, Del. The body is semi-convertible type, divided into two compartments, and has seating capacity for 35 passengers. The forward compartment contains the engine, and is arranged so that light baggage and express can be conveniently carried.

The engine is 50-60 H. P., heavy type, especially designed for this car and is built by the manufacturers of the car. Transmission is of the gear type, giving three speeds in each direction; changes are accomplished by sliding jaw clutches with the gears always in mesh and control is arranged so that it is impossible for the operator to throw in more than one speed at the same time.

Vibration is admirably taken care of by having the engine hung independently of the body of the car, the engine being placed on the truck frame while the car body is suspended on full elliptic

Gasoline Passenger Car.

springs. A single truck of special design is used so that it is possible to use a wheel base short enough to take severe curves. This truck has special equalizing spring suspension which permits a long body to be placed on a comparatively short truck with no oscilating movement. The car is equipped with air brakes operating on all four wheels.

The International Smelting & Refining Car is one of many of this type to be placed in passenger service. The performance of the other cars in operation has attracted much attention on account on their economy in operation which makes them a paying proposition on lines where passenger traffic is comparatively light.

The Manufacturer and Signal Maintainer

By F. Van Cleef.

It is a matter of common knowledge that the present state of railway signaling embodying its numerous intricacies and complications, working as a whole to facilitate a great railroad movement, is but an actual development of a few recent years. In that short span of time not alone have signals come into practice as one of the prime factors in the safe and speedy handling of trains, but as rapidly as they were introduced, equally rapidly has been their improvements and changes. These signal changes have increased the capabilities of the railroad and effected a larger number of train movements under certain track conditions; these changes have also improved the operations of the signals themselves, giving greater reliability of service, and besides this, they have improved the visual aspect and the indications are easier read. In all, better information is furnished to the enginemen and better service to the road. In doing this, however, much of the old simplicity has given way to a complication of device and circuits; they have failed to sufficiently consider labor conditions for the proper care and maintenance of the work, and in consequence that item has become of serious importance to the signal official.

The different designs of signal parts, mechanisms and circuits as applied to signal practice is usually the result of the ideas worked out by some trained signal man, and in the working out of these ideas the tendency is to introduce some complications beyond the grasp of the repairman or maintainer. The designers of the signal apparatus fail to consider that their natural aptitude to readily master a signal problem is probably the result of study, investigation and experience, so that the idea when evolved for some new signal device and ultimately installed, its maintenance and care becomes a serious problem in order that a continuous efficient degree of operation may be enjoyed. It is not the contention of the writer to imply that maintenance is forgotten, for we know that it is considered to a large degree, but facts at the same time show it is very difficult on many roads to secure men of sufficient training to meet the requirements.

Standardization of signal apparatus is a factor that in no little measure will aid the roads to co-operate with the situation, and probably goes far toward furnishing an interchange of labor. The Railway Signal Association, too, is a most important factor in assisting the railroads in the standardization of apparatus and circuits. Some manufacturers are in no small way helping the situation and great credit must be given them for the special devices which they are placing on the market that not alone assist in the operation of the signal systems, but furnish greater stability to the work, eliminate much of the unnecessary repairs, besides giving the entire installation a better lasting appearance. The art of signaling is so new that many opportunities are offered the manufacturer to bring out devices that will accomplish the above. This fact is obviously evident in one instance by the recent introduction of "trunking clamps," a new device, which embodies means for joining the wood-trunking to a metal clamp. It is something that gives a substantial support to the trunking at its weakest place, at the same time effecting true alignment of the work, an ease in installation and a permanent security that could not be obtained through any of the old methods.

Another novelty in apparatus recently brought out is the new form of lightning arresters without an air gap, which also relieves the maintenance situation. By the substitution of a very high resistance ground and the innovation of an automatic coupling unit, a lightning arrester for signal service has been brought out which is establishing a new standard in the art. Other items of equal importance which the details of benefits would be too lengthy to enumerate here are the "fiber steel battery chutes" and "pipe strain insulators."

The mentioned products are the introduction of the L. S. Brach Supply Company, 143 Liberty street, New York, who are manufacturers of signal specialties, bringing out many novel details, the result of effort to cope with the existing maintenance situation. In their products the Brach company does not attempt to change standard practice but to simplify the installing and maintaining conditions, offering many conveniences for the betterment of the signal system.

Political Status of the Railroads*

Recently a prominent railroad official said to me, regretfully: "The one thing which hurts me, Gray, is the fact that we are political pariahs. I have some legal ability, reasonable social position, have inherited and have made some money; I have never knowingly committed an act which I would fear for my parents to know, and yet because I have been a railroad and corporation counselor and employe, I cannot look forward, whatever may be my claims otherwise, to serving my country in an elective office."

The moving significance of this can best be appreciated when I say that this gentleman can trace his ancestry directly to one of the famous and noble families of the reign of the Tudors, is the son of a noted authoress, and his father was an officer of distinction favored with the close and intimate friendship of Jefferson Davis. Surely he represented what, in this land of democracy, might be termed the "social elect." Still young, a talented lawyer, unusually versed in the railroad business itself, with all that usually surrounds wealth and assured social position, yet he felt that politically he had the brand of Cain upon his brow, and that the erring daughter with the Scarlet Letter was not more vividly marked than he.

Generally speaking, this is unjust, but he has conned his life's lessons poorly who does not admit that there is a certain tinge of retributive justice in this splendid political isolation in which he and I and you, my railroad friends, find ourselves in this year of our Lord.

The first of the great corporations which now dominate every walk of commercial life were formed to build and operate the

*By C. R. Gray, Senior Vice-President St. L. & S. F. R. R., from St. Louis Railway Club Proceedings.

common carriers of this Union. Early they became, by virtue perhaps of their very uniqueness, the prey of the political buccaneer, and legislation was more frequently than otherwise sought for the sole purpose of mulcting the railroad into payments of ransom—sometimes money but generally transportation—often to individuals, but not infrequently to organized cliques in the very legislature itself.

Thus the railroads forced into political activity originally for the protection of their property, began to realize the power that went with an unbridled license and began to use that power, not as at first to defeat bad and unreasonable laws, but to stifle any legislation, bad or good, by which their powers were affected or abridged.

Then in his natural sequence came the freebooter, who, while ostensibly representing the railroads and blasting what remnant of reputation they still retained by the contamination of his evil genius, would introduce anti-railroad and corporation measures for the sole purpose of keeping the railroads still further in his power by his abilities to defeat these very measures. The railroads for years were at the mercy of these creatures, and to their shame be it said, but there is another side to this picture, the logic of which will appeal to the open-minded student of affairs, and which is that the methods themselves, and the loathsome instrumentalities, at first used by the railroads, but which afterwards became their masters, would have been abortive in the extreme if the majority of the men responsible for the legislation had not been of like kidney, and the proof of this lies in the fact that when a healthier moral tone began to pervade public affairs, there was no one to whom the innovation was more truly welcome than to the railroads themselves. In this respect at least the railroads are entitled to favorable comparison, for at their worst politically they were as good as the agencies with which they dealt; both were tarred with the same brush.

Gen. Lew Wallace once told of a visit he made to a small town in Indiana. As he drove up from the station with the usual loquacious cab driver he noticed an extraordinary fine church and on inquiry was told that it was a Congregational church.

A block farther on there was another church an exact replica of the first; either an astonishingly good church for the town, and he was assured that this also was a Congregational church. Upon pressing the cabman as to the distinction between the two, he was informed that the first church believed that Eve tempted Adam, and the second church believed that Adam was a "bad egg" from the start.

The railroad lobbyist (using the term in its offensive sense) is a thing of the past. The designation, once a term of reproach, will in the future be proudly borne by thousands of men who will represent sturdily the rights of their properties before the assemblies of the people's elect, and who will, I am serenely confident, command respect and attention in the degree that they respect themselves.

Years of right dealing in this respect alone can rehabilitate the railroads in the public estimation, but there can be no doubt that this will be done. A far-sighted railroad official today desires nothing more earnestly than a square accounting with his vast clientele—the public—and the honest and efficient administration of laws which protect as well as punish.

Measured by correct political standards the railroad man of the past has not risen to his opportunities, but we must live in the present and should face hopefully toward the future.

A short while ago a play was presented in this city called the "Melting Pot." The conception from which the play took its name arose from the beautiful fancy of a poor Russian Jew—a political refugee who had found a haven in our land. His dream was that our country was the world's vast melting pot into which were poured for admixture and solution the varied races of the earth.

Into this crucible came all sorts and conditions of men—Jew and Gentile, Latin and Goth, Slav and Saxon, passing under the spell of the great welcoming statue of liberty and emerging from this cauldron, purified by fire, regenerated by infusion and association, grown strong through civil and religious freedom, to become *the* American of the times that is to be.

In his scantily furnished apartments he had found place for pictures of the Father of his Country and of the Great Emancipator and a small flag, the banner of the stars.

Somehow the wild and unquestioning loyalty of this poor Jewish boy made one ashamed that we take calmly and dispassionately the glorious freedom which so intoxicated him.

The railroad man, too, has an abiding interest in this Melting Pot, for in the years that are to come it will mean much to him that the Great Chemist has wisely distributed His portions, for many of the great army of railroad employes must be recruited from this source, and it is from their character as well that the future must decide whether the political atrophy which has afflicted our generation is to endure.

Every railroad employing-official ought to feel the great responsibility that inevitably goes with the selection of men. Perfection in this will never be attained, but should be always sought. The men who never make mistakes are those who attempt nothing. Fortunate are we indeed to be generally right, and certain are we indeed to be often wrong.

Careful selections are oftentimes thwarted by conditions which could not have been humanly foreseen, and it should comfort that employer smarting under a particularly aggravating failure in this respect that the First gentleman of the World, the Savior of mankind, was similarly afflicted in His chosen twelve.

During the Civil War an Alabama regiment had been drawn up preparatory to making a desperate charge. A small cottontail rabbit started in desperate haste for the rear. One of the soldiers looking after the rabbit wistfully cried out, "Go it, Molly; I'd be with you if it warn't for my char-racter."

Character! What a center shot that rough soldier fired, for if I were asked what in my opinion is most needed to elevate and enfranchise the railroad profession, it is contained in that one word Character, and in this I do not have in mind only the fortunate few who attain official preferment.

To quote former President Roosevelt:

"The only safety in our American life lies in spurning the accidental distinctions which sunder one man from another, and in paying homage to each man only because of what he essentially is; in stripping off the husks of occupation, of position, of accident, until the soul stands forth revealed, and we know the man only because of his worth as a man."

There is not a railroad employe who has not his circle of influence and in which his railroad is good or bad as he reflects those attributes in his daily life.

It is an historical fact that enduring success can not be attained by any cause to which wrong attaches. There is some reason to believe that the Confederacy would have won a place in the galaxy of nations if its cause had not been irrevocably bound up in the world's opinion with the bane of human slavery, and all the bravery of her sons, and all the blundering of the early Northern generals could not accomplish anything of a permanent nature, because across the brilliancy of the Southern Cross was inevitably thrown the shadow of this blighting institution. And yet not one in a hundred of the soldiers of the Confederacy had ever owned a slave or ever expected to.

In the last analysis the situation calls for a higher moral plane, for cleanness in dealing with public questions (and practically every problem we have is of public importance) and this need is even greater than the natural desire for a higher order of intelligence, for it is inevitable that the latter condition will follow once the first is established.

We must as well be adaptable, and must meet conditions as they exist and not as we think they should be. There should be no tilting at windmills.

Before the war there was a fire-eater in Georgia who in his speeches invariably insisted that the South could whip the Yankee with popguns. He disappeared during the war, but after it closed was back and again took the rostrum. At one of his

early speeches, replete with superlatives as before, an old soldier who had seen desperate service throughout the entire conflict, said, "Look here, ain't you the same feller that told us before the war we could lick the Yankees with popguns?"

The reply came instantly:

"Certainly, and we could, too, but darn 'em they wouldn't fight us that way."

It remains, therefore, that to secure respect (for with respect will come the kindred attributes of regard and affection) the railroad man of the present and future must so conduct himself as to silence doubt, confute slander, and walking, as he does, in the full noon of public knowledge, justify by his private and official life the fullest confidence in the probity and morality of his official motives, and then—and not until then—will he be taken at his own estimation of himself.

I know of no better thought in closing than to quote the Immortal Railsplitter, whose words seem doubly significant when read in this connection:

"I am not bound to win, but I am bound to be true.

"I am not bound to succeed, but I am bound to live up to what light I have.

"I must stand with anybody that stands right; stand with him while he is right, and part with him when he goes wrong."

Increasing Forest Productivity

Proper management of any forest necessitates complete protection from fire, and a market in which wood material of all sizes is salable at a price sufficient to make the practice of intensive forestry profitable. Both of these conditions will prevail in this country in the future. Therefore, they are assumed in this paper.

Productiveness will be treated as the equivalent of growth, for in a properly managed forest the cut is equal to the growth or increase in wood material. In our forests the cut exceeds the present growth, or production of wood by the forest, for we are now cutting annually the growth of many past years. In the near future the cut cannot increase unless the growth or productiveness of our forests increase as well.

Increase in both the quantity and the quality of the product of our forests can be secured through proper management. The increase in quantity will result from an increased area of producing forest on absolute forest land, and from having this area stocked with those species of trees which produce in the shortest time the classes of material most in demand. The increase in quality will result from managing these stands so that material of desirable size and shape is produced.

Production From Forests Now Virgin or Mature.

When settlement began, the forests of the United States were not producing forests, nor are the yet uncut and unburned areas producing to-day. There are growing trees in all our mature or virgin forests, but their growth is merely enough to offset loss from natural causes, such as decay, windfall, lightning and insects. The forest has produced, and is now merely keeping the amount of its crop at the point reached years, perhaps centuries, ago. It is forest ready for cutting and can be made producing forest only by cutting in a way that will encourage a new crop to grow.

Ultimately the present virgin or mature forests will be cut over, and the land used either for agricultural crops or for new crops of timber. Under proper management the present condition of unproductiveness will never occur again, for the timber crop on any area kept in forest will be cut as soon as, and usually before, it has stopped increasing. The ideal toward which the forester aims may be illustrated by assuming that 100 years are required for a timber crop to become mature, that an area of 100,000 acres is to be handled as one forest, and that each cutting is followed immediately by reproduction. The forest would then be divided into a series of 100 stands, each of 1,000 acres, differing in age by one year, from the yearling trees up to the century old mature timber. Each year the stand which has reached the century mark would be cut, and the crop from that stand of 1,000 acres would be equivalent to the growth for that year on the whole tract of 100,000 acres. Thus the whole forest would be continuously productive.

In our virgin forests it is as if each stand on reaching the age of 100 years had remained stationary until the whole forest is composed of mature timber. There is no increase in the amount of timber on the ground and consequently no productiveness.

In our virgin forests it is usually both unnecessary and undesirable to cut clean and to establish a new crop by artificial means, in order to make the areas productive. In most of our forests there are young trees already present in sufficient numbers to form the basis for a second crop when the trees now mature are cut, and these young trees will grow the faster when they receive increased light and, in the drier portions of the country, increased soil moisture, after a conservative cutting.

There are approximately 188,000,000 acres of mature forest, exclusive of woodland, in the United States, not one acre of which can be classed as producing forest. Some of this land is more valuable for agricultural crops than for growing timber, and will be put under cultivation as soon as cleared. A conservative estimate, however, is that 135,000,000 acres of the present virgin or mature forest are more valuable for growing forest crops than for any other purpose, and may become productive forest land.

Production From Waste Lands.

There is in the United States another class of lands which has no productiveness at present. Worn-out agricultural soils, cut and burned areas which are not restocking, and sand dunes, are examples of what are commonly called "waste lands." Experiments have shown that some of these areas need only protection from fire to become productive. Any of them can be artificially stocked with forest trees, and so made productive, for forests can grow on soils too poor for successful agriculture, and, as a rule, should be allowed to grow only on such soils. Also, it is extremely probable that future experiments will show it practicable to grow forests on land too dry, too acid, or too alkaline for farming.

There are 81,000,000 acres of forest land, exclusive of woodland, which have been cut or burned over, but are not restocking. Possibly 10,000,000 acres can be used for agriculture, but additions from worn-out agricultural soils, sand dunes, and other sources give a total of at least 90,000,000 acres of waste lands in the United States, all of which can be made productive by artificial stocking with forest trees, or in some cases by merely giving protection from fire.

In making this conservative estimate no consideration has been given to the increasingly large areas of plantations on soils which might be classed as agricultural, but which are devoted to tree production for the sake of wind-breaks or farm wood lots. These are not waste lands, but may be classed with them, for they are areas which are not now, but in the future will be, productive forest.

Possible Average Production Per Acre.

The present growth of fully stocked, growing stands ranges from 30 to 110 cubic feet per acre per annum, with an average of 70 cubic feet for the whole country. The actual production per acre is, however, much less, due chiefly to the absence of fully stocked stands, and when applied to the small proportion of our forest land which is now productive gives a total annual production of only 6,500,000,000 cubic feet.

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*By E. E. Carter, Assistant Forester, Forest Service.

The estimated growth per acre of fully stocked stands is already higher than for European countries, for our forests as a whole are in regions extremely favorable for tree growth. As an illustration, on over 75 per cent of our forest area the annual rainfall exceeds 35 inches, and in some localities is over 100 inches. Also, large areas of our forests are already covered with fast-growing species, such as white pine, which, on fair sites, shows a mean annual growth at the age of 60 years of over 140 cubic feet, or double the average for the country.

Any attempt to predict an increase in the growth in cubic feet of fully stocked stands for the country must be purely an estimate. Such an increase is possible, however, and a conservative figure for it is 10 cubic feet per acre, making a total annual production of 80 cubic feet per acre.

Total Increase in Quantity Possible.

The possible increase in growth in cubic feet is equivalent to an annual growth of 80 cubic feet on the 135,000,000 acres of our virgin or mature forests, on the 90,000,000 acres of our waste lands, and on the 80,000,000 acres of land unproductive on account of lack of density in our producing forests. This gives a total of 24,400,000,000 cubic feet. To this must be added an increased growth of 10 cubic feet per acre on 145,000,000 acres (the area on which our present producing forest could be grown), or 1,450,000,000 cubic feet. This makes the total possible increase 25,850,000,000 cubic feet.

Summary.

There are 225,000,000 acres of producing forest in the country. On this area the stand is so open that all the trees could be grown on 145,000,000 acres, which would be producing at the rate of 70 cubic feet per acre annually. By growing only the best species this growth can be increased to 80 cubic feet, an increase of 1,450,000,000 cubic feet.

There is an unutilized area of 80,000,000 acres within our present producing forest. There are 135,000,000 acres of absolute forest land within our virgin or mature forests now unproductive. There are 90,000,000 acres of waste lands which can be made productive by planting or fire protection. All of these areas can be made to produce annually an average of 80 cubic feet per acre, or a total of 24,400,000,000 cubic feet.

The total increase possible in the productiveness of our forests is, therefore, 25,850,000,000 cubic feet.

Only 25 per cent of the yield of our present producing forest is saw timber. It can be made from 50 per cent to 75 per cent saw timber through the concentration of growth on the best trees by thinnings, and the holding of the crop till it reaches good size. This would mean an increase, if all our forest land were productive, from 27,000,000,000 board feet to between 54,000,000,000 and 72,000,000,000 board feet per annum.

Personals

Mr. W. G. Massenburg, chief engineer of the Gulf & Interstate at Galveston, Tex., whose appointment as division engineer of the Gulf, Colorado & Santa Fe, with office at Beaumont, Tex., has been announced, will act as a joint employe of both companies.

Mr. S. W. Massey, roadmaster of the San Luis division of the National Railways of Mexico at San Luis Potosi, Mexico, has been transferred to the Mexico Queretaro division, with office at San Luis Potosi.

Mr. H. M. Stone, district engineer of the Northwestern district of the Chicago, Rock Island & Pacific at Cedar Rapids, Ia., has been transferred to the Southwestern district, with office at Topeka, Kan., succeeding Mr. J. G. Bloom, resigned to go into other business. Mr. Roy Johnson succeeds Mr. Stone, with office at Cedar Rapids.

Mr. Wm. Hood, chief engineer of the Southern Pacific, with office at San Francisco, Cal., has had his jurisdiction extended over the Gila Valley, Globe & Northern, the Maricopa & Phoenix, the Phoenix & Eastern, the Arziona & Colorado and the Arizona Eastern. Mr. L. H. Long, chief engineer of the Southern Pacific Railroad in Mexico, with office at Tucson, Ariz., has been appointed also assistant chief engineer of the above roads.

Mr. L. C. Heilman has been appointed supervisor of signals at Cedar Rapids, Ia., having jurisdiction over the Northern district of the Chicago, Rock Island & Pacific, vice Mr. G. W. Trout, transferred to Trenton, Mo., as acting supervisor of signals of the Missouri division.

Mr. H. E. Henderson has been appointed to the new position of superintendent of bridges and buildings of the Pan-American.

Mr. J. B. Berry, supervising engineer of the St. Louis & San Francisco, with office at St. Louis, Mo., having resigned, that office is abolished. The title of Mr. C. M. Byers, chief engineer, will be changed to chief engineer-operation.

Mr. J. G. June, assistant engineer of the Erie at Jersey City, N. J., has been appointed engineer of the Buffalo division and branches, with office at Buffalo, N. Y., succeeding Mr. R. P. Madill, resigned.

Mr. R. C. Gowdy, resident engineer of the Fort Worth & Denver City at Fort Worth, Tex., has been appointed chief engineer, with office at Fort Worth, and chief engineer also of the Wichita Valley, with office at Wichita Falls, Tex.

Mr. W. J. Muff has been appointed a roadmaster of the Atchison, Topeka & Santa Fe, with office at Florence, Kan., succeeding Mr. F. Powers. Mr. William Turner, recently appointed roadmaster of the Oklahoma division, with office at Guthrie, Okla., has been transferred to Arkansas City, Kan., Mr. H. S. Cox succeeding Mr. Turner. Mr. E. B. Dehart has been appointed roadmaster of the Colorado division, with office at Pueblo, Colo., succeeding Mr. Wm. Davies. Mr. D. Bogue has been appointed roadmaster of the Mexico division, with office at Trinidad, Colo., succeeding Mr. Charles Ennis. Mr. R. R. Black, assistant roadmaster on the Rio Grande division at Vaughn, N. Mex., has been appointed roadmaster, with office at Vaughn, succeeding Mr. T. G. McNeill. Mr. E. F. Fleming, recently appointed roadmaster of the Rio Grande division, with office at Rincon, N. Mex., succeeds Mr. Black. Mr. C. L. McCallister, roadmaster of the Rio Grande division at San Marcial, N. Mex., succeeds Mr. Fleming, and Mr. P. Brumbaugh succeeds Mr. McCallister. Mr. L. H. McIntire has been appointed roadmaster of the Oklahoma division, with office at Guthrie, Okla., succeeding Mr. J. E. Markle.

Mr. Alfred Noble has severed his connection with the Pennsylvania Railroad as chief engineer of the East River division of its New York tunnel extension. Since 1902 Mr. Noble has been a member of the Board of Engineers appointed by the late A. J. Cassatt, president of the company, to examine its proposed New York tunnel extension project, pass upon the practicability of the undertaking, and determine upon the best plans for carrying it out. When this work was authorized Mr. Noble was appointed chief engineer of the East River division.

Trade Notes

The Watson-Stillman Company has made several additions to its sales department to handle the increasing business in hydraulic tools and turbine pumps. Edwin Stillman has entered the sales department, and is assisting in take care of customers in New York state, while all southern railroad business is now in charge of Frank C. Clark. The more direct representation that has become necessary in the Orient will be in the hands of F. W. Horn, the well-known machinery importer of Yokahama, Japan.

Mr. A. D. McAdam, who resigned as vice-president of the Damascus Brake Beam Co., has been elected vice-president of the St. Louis Surfacer and Paint Co., with headquarters at 1101 Fisher building, Chicago, and will devote practically his entire time and attention to the rapidly growing business of said company with the railroads of Chicago and vicinity.

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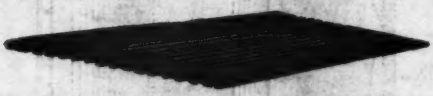

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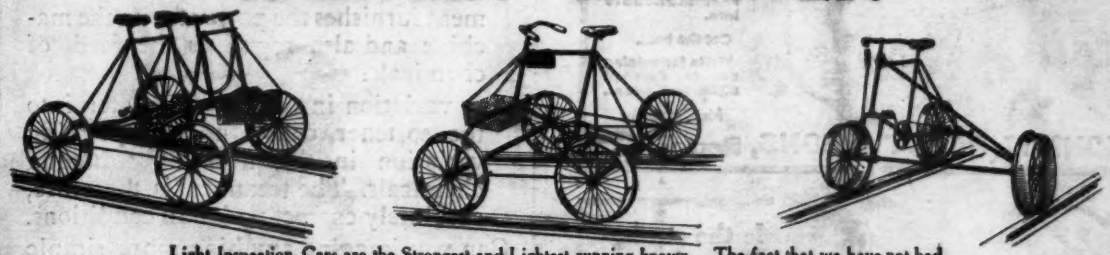

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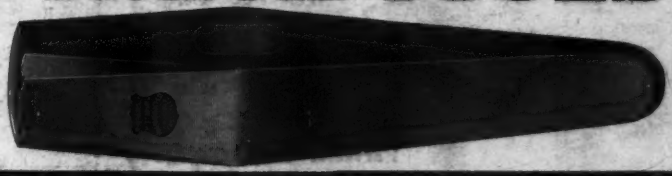

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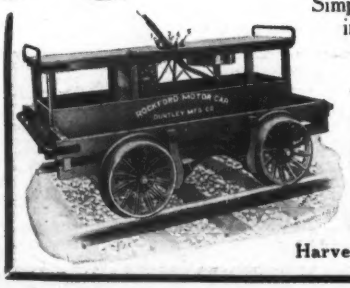

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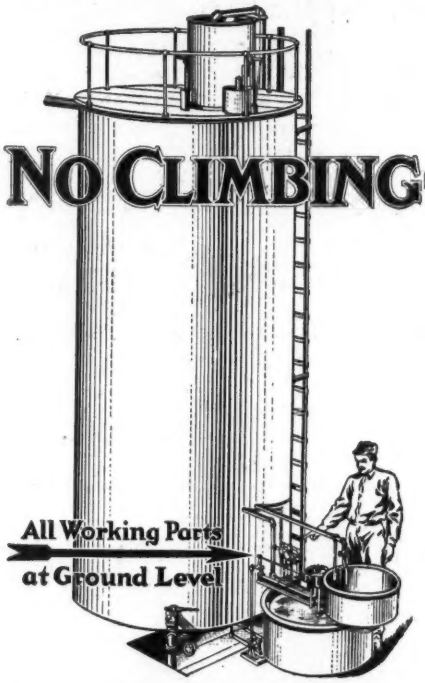

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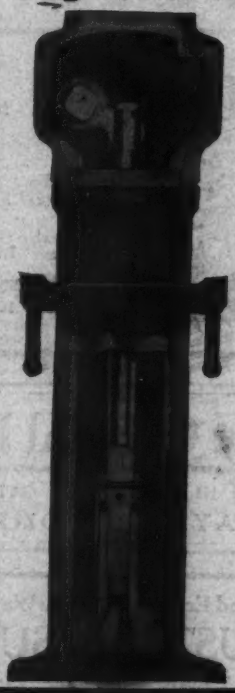

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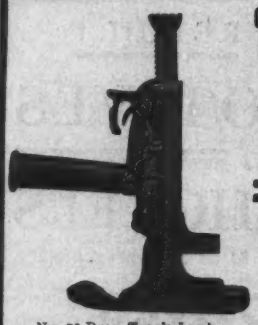

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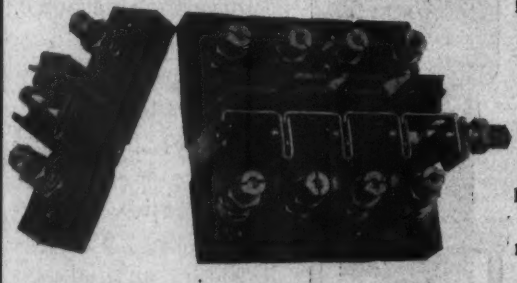

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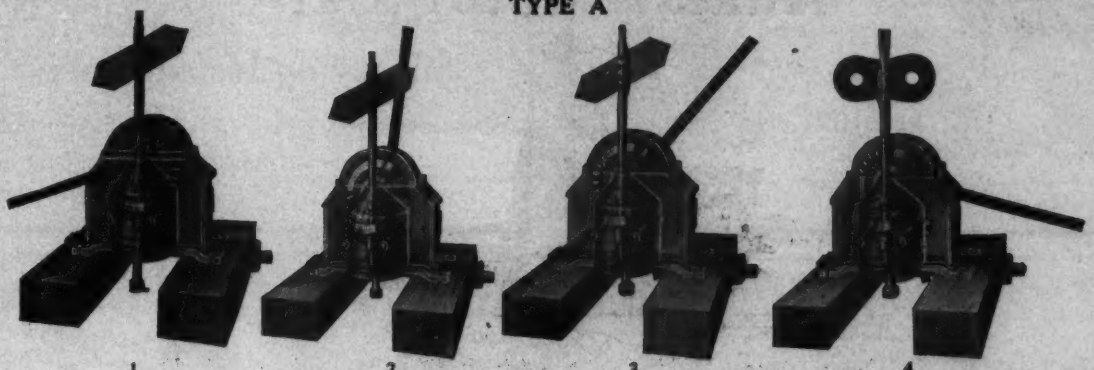

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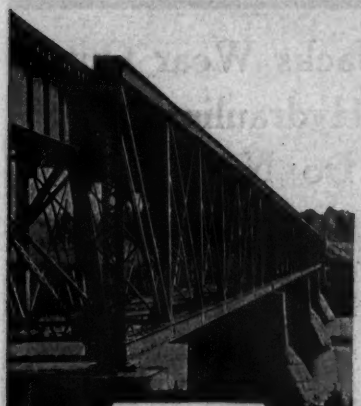

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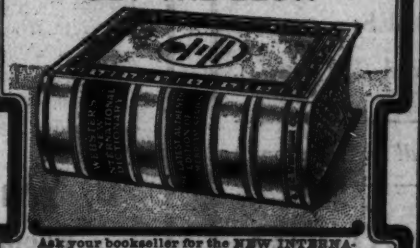

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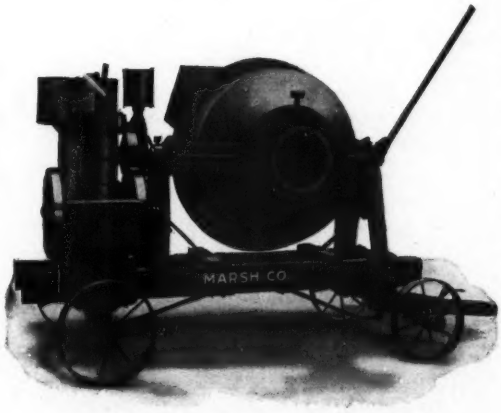

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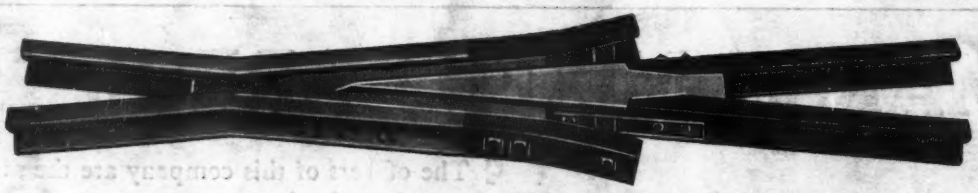

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